THE UNIVERSITY OF ALBERTA

HYDRAULIC INVESTIGATIONS RELATED TO LARGE CORRUGATED-METAL CULVERTS

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE IN CIVIL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

by

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FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled "Hydraulic Investigations Related to Large Corrugated-Metal Culverts", submitted by Charles R. Neill in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

ABSTRACT

Previous research on the hydraulics of culverts is reviewed and its findings compared with existing design information and practice. Field experiments on a 60" culvert are described and their results discussed with reference to previous research, design publications, model experiments, and hydraulic theory. A new form of chart for the hydraulic design of corrugated-metal culverts is presented. A few problems in fluid mechanics arising from the experiments are discussed. Recommendations are made concerning the design of highway culverts, further practical experiments, and basic research.

ACKNOWLEDGEMENTS

The author wishes to express his thanks to the many people who co-operated in the research project on which this report is based; to the Research Council of Alberta, the Alberta department of Highways, and the University of Alberta, who supported the project under the province's Joint Highways Research Program; to Dr. T. Blench, Professor of Hydraulics at the University, who took a keen interest in the project and gave much valuable advice and criticism; to Mr. E.J. Sanden and other engineers of the Bridge Branch, Alberta Department of Highways, who suggested the project, designed and built the field experimental installation, and assisted in the experiments; to Mr. W. Slater, manager of the Western Irrigation District at Strathmore, Alberta, who arranged the generous co-operation of the District which made the experiments possible; and to the hydraulics staff of the University of Alberta, who gave suggestions and assistance.

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CHAPTER 1 - INTRODUCTION

1.1. THE PROBLEM

In recent years the Alberta Highways Department has installed a large number of corrugated-metal culverts. The engineers of the Bridge Branch have felt for some time that the available information for their hydraulic design was inadequate and inaccurate, and that recent research had perhaps discovered improved design features which should be incorporated in future construction. They therefore initiated this investigation, which comprised field and laboratory experiments, a search of existing literature, and theoretical studies.

1.2. FEATURES OF CORRUGATED-METAL CULVERTS

Fig. 1 shows a profile of a typical highway culvert, together with various types of inlet which will be discussed. Some of these are given different names by other writers, but the descriptions used in Fig. 1 will be used consistently throughout the report.

Most of the large corrugated-metal culverts in Alberta have bevel flush inlets, with outlets of the same form, although square projecting ends are sometimes used. Bevel ends are used

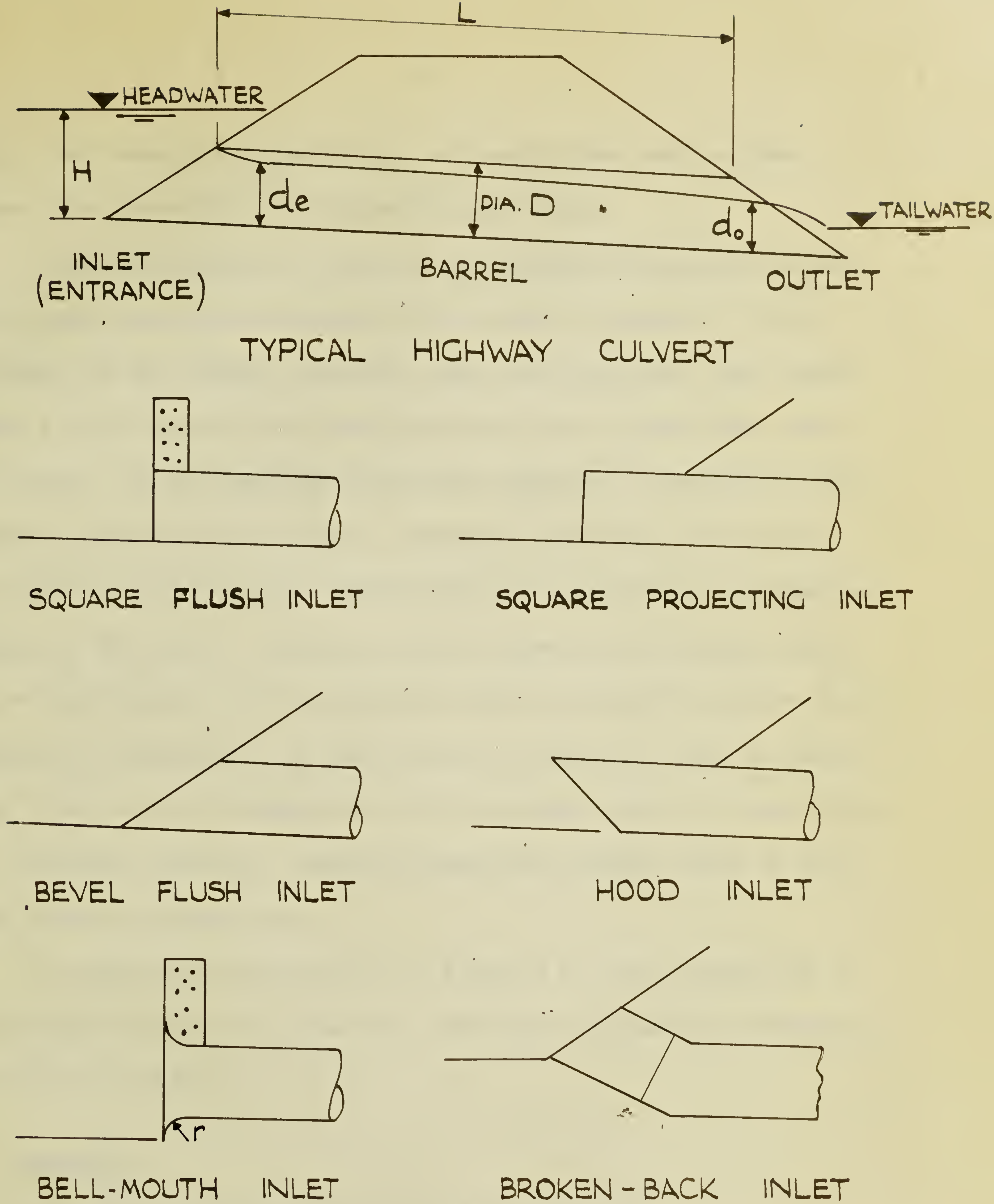

FIG. 1 - TYPICAL CULVERT DETAILS

mainly for reasons of appearance, although there was an idea at one time that they had hydraulic advantages.

Corrugated-metal is used in preference to concrete partly for economy, and partly because it is quick to instal. It is available in two shapes, circular pipe and pipe-arch, the latter having a flat invert and a smaller rise, for a given area, than the circle. It is fabricated from two types of corrugated plate, standard and structural plate. Standard plate has corrugations 1/2" deep at 2-2/3" pitch, while structural plate has corrugations 2" deep at 6" pitch. Standard plate is generally rivetted into pipe at the factory, but structural plate is bolted together from segments in the field. Standard pipe is generally used in sizes up to 5 ft. or so in diameter, and structural plate in sizes from 5 ft. diameter upwards. Nominal dimensions always refer to net sizes inside corrugations.

It has been the practice in Alberta to lay culverts on a longitudinal camber, to allow for settlement ultimately producing a straight profile.

1.3. NOTATION

The following notation is used throughout:

A = cross-sectional area of flow

B = breadth of water surface

C_d = coefficient of discharge (for orifice or weir)

C.M. = corrugated metal

d = depth of flow (with suffixes)

D = culvert diameter or vertical dimension

E_f = energy of flow

f = friction factor

g = gravitational acceleration

h = head loss (with suffixes)

H = headwater depth (see Fig. 1)

H.G.L. = hydraulic grade line (piezometric head line)

H_L = total head drop across culvert flowing full

K = head loss coefficient (with suffixes)

K.E. = kinetic energy

L = net length of culvert (see Fig. 1)

n = Manning's roughness coefficient

P = wetted perimeter of cross-section

Q = discharge

q = discharge per unit width (rectangular channel)

R = hydraulic radius = A/P

R_n = Reynold's number

r = radius of bellmouth

s = slope

s_o = slope of bed or invert

T = tailwater depth

V = velocity

α = kinetic energy correction factor

Suffixes: e refers to inlet (K_e, d_e etc.)

o refers to outlet (K_o, d_o etc.)

c refers to critical flow conditions (V_c, d_c etc.)

f refers to friction (h_f, s_f)

m = mean (V_m, R_m etc.)

Lb. ft. sec. units are used throughout.

All dimensions are net inside corrugations.

Certain other symbols are defined as they arise.

1.4. DEFINITIONS

Familiarity with conventional pipe and open channel theory is assumed, but a few definitions, referring mainly to open channels, are given below:

Critical Conditions in a conduit flowing with a free water surface occur when the velocity is just sufficient to prevent small surface waves or disturbances from travelling upstream.

Critical Velocity is the mean velocity at critical conditions.

Super-critical Flow means flow at a velocity greater than critical.

Sub-critical Flow means flow at a velocity less than critical.

Critical Depth, for a given discharge, is the depth at critical conditions. It is a function of the geometry of the channel cross-section only.

Kinetic Energy Correction Factor, applied to the velocity head based on the mean velocity ($V_m^2/2g$), corrects for non-uniform velocity distribution across the section to give the true average velocity head.

Energy of Flow = depth + velocity head = $d + \alpha V_m^2/2g$.

Normal Depth for a given discharge and slope is the depth at which this discharge will flow uniformly, and is found by trial applications of Manning's formula.

Critical Slope for a given discharge is the slope at which normal depth = critical depth.

Steep Slope is any slope exceeding critical. On steep slopes, normal depth is less than critical depth.

Mild Slope is any slope less than critical. On mild slopes, normal depth is greater than critical depth.

Backwater Curve means any water-surface profile not parallel to the bed or invert of the channel, i.e., flowing non-uniformly.

Friction Slope for a given discharge and depth is the slope for uniform flow, and is found by applying Manning's formula.

Free Outlet (of a culvert) means that the tailwater is not sufficiently high to cause a backwater effect in the barrel.

1.5. THE CULVERT AS A HYDRAULIC DEVICE

Elementary hydraulics texts deal with such devices as weirs, orifices, open channels, and pipes. A culvert is more or less a combination of all of those. In the applications with which this report is mainly concerned, a culvert functions generally like an orifice followed by a pipe or open channel. It is this interaction of different types of flow which makes the hydraulics of culverts far from simple.

CHAPTER 2 - REVIEW OF LITERATURE

This chapter is concerned with culverts in general, and not only C.M. ones.

2.1. HISTORICAL REVIEW OF RESEARCH ON CULVERT HYDRAULICS

All the work described was done in the U.S.A. Letters from research organizations in Canada and the United Kingdom indicated that no work had been done on culvert hydraulics in either country. No foreign-language sources were consulted.

2.1.1. The earliest investigation of any importance was that of Yarnell, Nagler and Woodward(1) in 1926. The experiments were extensive, and consisted mainly of determining discharges through a series of concrete, tile, and corrugated-metal culverts set up in a laboratory. Pipe culverts were up to 30" diameter, and box culverts up to 4' square. Various types of inlet and outlet, and various arrangements of headwalls and wingwalls, were used. Slopes varied from zero to 7%. All the culverts were about 30' long.

The theory of culvert operation was not too well understood at this time, so that the interpretation of the data was not too satisfactory, and the design information was somewhat misleading. In most of the experiments the culverts were operated so that they

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ran full, and it does not seem to have been realized that in field conditions most culverts did not run full.

2.1.2. Mavis[2] (1943) realized that the problem was much more complex than had been thought in 1926, and listed 9 variables and 5 modes of flow. He emphasized the type of flow which often occurs in practice, when the inlet is submerged but the culvert flows only part-full, the inlet acting as an orifice and controlling the discharge. He experimented on pipes 3" and 6" diameter by 4.4' long, and 12" diameter by 12' long, with square flush inlets, and plotted the ratio H/D against the ratio $Q/D^{5/2}$, to give a non-dimensional rating curve. The points for the various sizes plotted extremely close together, and a single curve was drawn through them. This curve, which is reproduced in Fig. 2, was then used as the basis for a design nomograph, applicable to culverts of any size.

Mavis' rating curve has been widely referred to by later investigators, and its application to culverts of any size under inlet control conditions has never been seriously challenged. Mavis did not offer any theoretical explanation of it. He made it clear that it applied only to pipes with free outlets on steep slopes.

2.1.3. Larson and Morris[3] (1948) did no experimental work in connection with their project, which consisted mainly of an exhaustive review of all available literature bearing on the subject,

and an attempt to present a comprehensive theory and methods for design. The annotated bibliography listed 118 references on hydraulics, as well as others on hydrology and structures, and the report contained 53 abstracts.

In addition to full-flow operation, as in the 1926 experiments, and inlet control operation, as in Mavis', the authors recognized another mode of operation, part-full flow with a submerged inlet on a mild slope, but only touched on a method of analysis. For the design of culverts on steep slopes, they recommended the use of Mavis' curve, but had reservations about its range of applicability.

2.1.4. Straub and Morris[4] (1950) described work carried out under the sponsorship of concrete pipe and cement organizations, to determine values of the entrance loss coefficient and the roughness coefficient for concrete and C.M. pipe under various conditions. Their pipes were from 18" to 36" diameter and approximately 200' long.

2.1.5. Shoemaker and Clayton[5] (1953) investigated box culverts and methods of improving their operation by means of new inlet designs, using a model 4" square by 6' long, which was tested at slopes of up to 8%, and with 7 different designs of inlet. They found, as Mavis had found with pipes, that box culverts with plain inlets, whether or not provided with wingwalls, did not flow full

on steep slopes. With a laterally convergent inlet, however, the model flowed full when H/D exceeded about 1.2, with consequent increase in discharge. It was suggested that existing box culverts could easily be improved by constructing a roof over the wingwalls, to form an inlet of the type tested.

The experimental results were presented as a series of rating curves and hydraulic grade line profiles. The authors warned that they should not be scaled to predict prototype performance.

2.1.6. Straub, Anderson and Bowers[6] (1953) described experiments on a 4" diameter pipe 35' long at a series of slopes up to 10%, with free outlets. The main object was to demonstrate the increase in capacity which could be obtained by replacing the square flush inlet by a bellmouth inlet with an r/D ratio of 0.15. They found that, whereas the square inlet caused contraction of flow through the inlet and part-full flow in the culvert, as noted by earlier investigators, the bellmouth inlet caused the pipe to flow full for H/D ratios exceeding about 1.2 to 1.5. At low depths of submergence, full flow did not appear to be stable, the culvert tending to show an oscillating condition called "slug flow".

The paper contained an extensive analysis of various modes of flow in culverts. It was pointed out, that in the case of culverts on mild slopes, the advantage to be gained by using a bellmouth

of other tubes). With a laterally [illegible] flume, however, the model showed full capacity [illegible] about 1.7. With [illegible] increase in discharge. It was suggested that [illegible] the outlets could easily be improved by [illegible] a [illegible] near the [illegible], to act as inlet of the pipe [illegible].

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inlet was not nearly so great as with steep slopes. The non-dimensional rating curve presented for the square flush inlet agreed very closely with Mavis' curve.

2.1.7. Karr and Clayton(7) (1954) described model studies of inlets for pipe culverts on steep slopes. The model was 4" diameter, by 7' long, and was on a 4% slope for most of the tests. The inlets tested included, among others, two designs of hood. The hood inlet was developed as a simple device to make the pipe flow full under submerged inlet conditions, thus performing the same function as a bellmouth inlet. Both designs of hood appeared to work satisfactorily. No theoretical explanation of their operation was offered.

2.1.8. Li and Patterson(8) (1956) described experiments on small models of box and pipe culverts. They were mainly concerned with "self-priming", by which they meant the tendency of culverts with square inlets to flow full under certain conditions. Charts were presented which, it was claimed, enabled those conditions to be predicted. Data were also presented on the location of the hydraulic grade line at free outlets, for pipes flowing full.

In the discussion of this paper, Blaisdell asserted that the so-called self-priming was essentially unstable, and that the charts were of no value.

2.1.9. <u>Carstens and Holt</u>[9] (1956) presented a series of photographs illustrating a wide variety of flow conditions in a small two-dimensional model. While they are of some value in obtaining an understanding of the complexity of the problem, it is questionable how closely they represent conditions in an actual culvert.

2.1.10. <u>Schiller</u>[10] (1956) experimented on inlets for pipe culverts, using a 5" diameter model about 5' long. The inlets tested included a thin-walled square projecting, a bevel flush, a thick-walled bevel with rounded edges, a square flush, a bell-mouth, and an inlet cut to model the groove end of concrete pipe.

The experimental results were presented as non-dimensional rating curves. Only the bellmouth and the groove inlets caused full flow, and it is not clear whether the success of the groove inlet was due to testing it only on a mild slope. The other inlets all produced curves similar to Mavis', with small variations. It appeared that, with thin-walled pipe, there was no advantage in using a bevel inlet rather than a square projecting. Schiller's curve for the square projecting inlet is shown together with Mavis' curve in Fig. 2. His curve for a square flush inlet is practically identical with Mavis', and that for the bevel flush inlet is practically the same as for the square projecting.

The experimental results were used to prepare design nomographs, applicable to concrete and C.M. pipes on steep slopes, with

2.2.9. [illegible] (1956) presented a series of photographs illustrating a wide variety of flow patterns in a small two-dimensional model. [illegible] of the [illegible] of the [illegible]. It is unfortunate that [illegible].

2.2.10. [illegible] (1956) [illegible]

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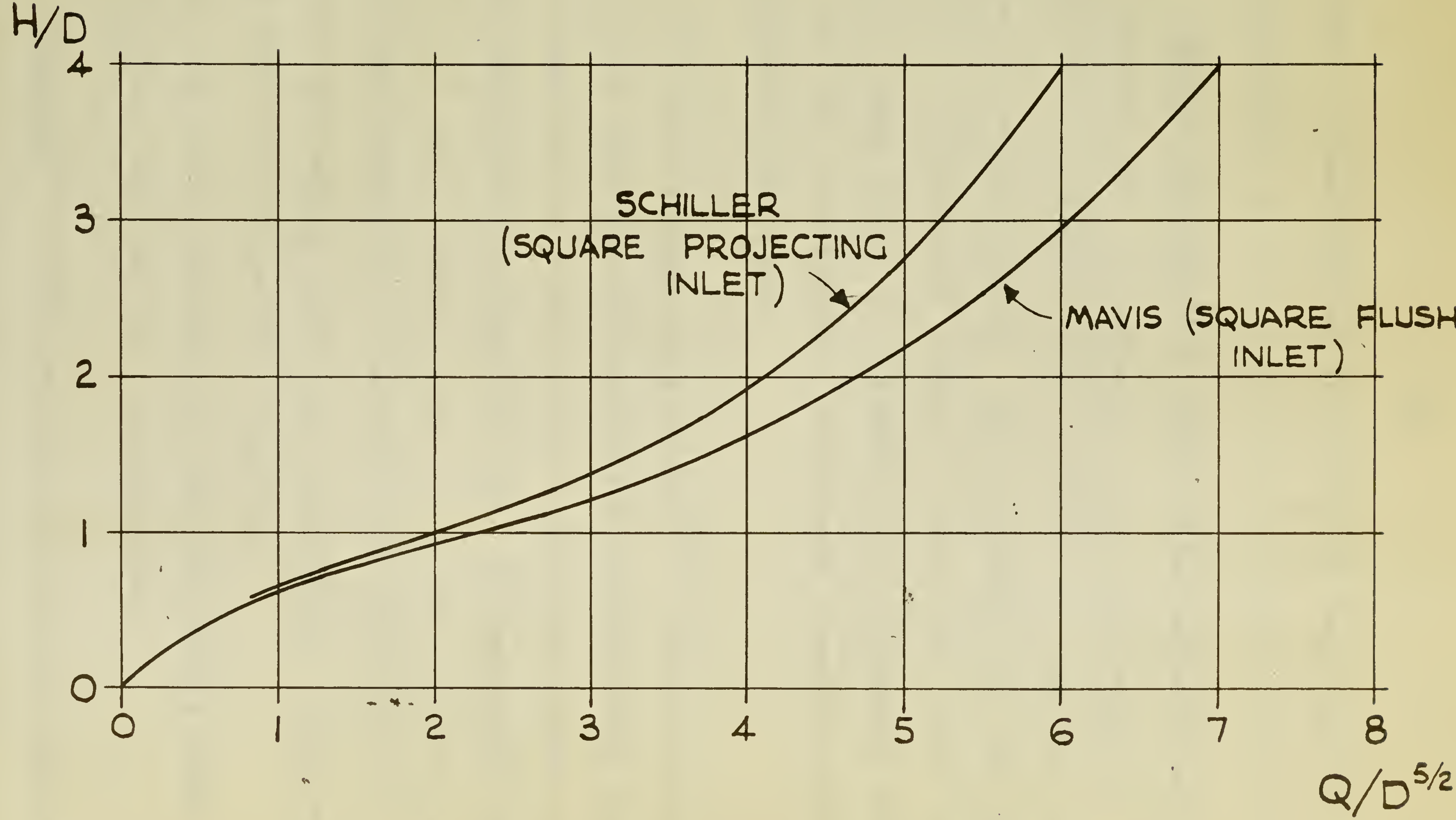


FIG. 2 - NON-DIMENSIONAL RATING CURVES FOR PIPE CULVERTS ON STEEP SLOPES (FROM MODELS)

free outlets. A nomograph was also given for both types of pipe flowing full.

2.1.11. Metzler and Rouse[11] (1959) did not actually deal with any new research work, but presented a very lucid summary of hydraulic theory as applied to box culverts, and summarized the conclusions of recent research. They recommended for use, where practicable, a "broken-back" design of inlet, with lateral convergence, as shown in Fig. 1, claiming that it eliminated unstable flow at low heads of submergence, and ensured a smooth transition from part-full to full flow.

2.1.12. The work of Webster and Metcalf[12] (1959) on friction factors in C.M. pipe, was not specifically concerned with culverts, but is of importance in view of previous uncertainty about the roughness coefficient for large C.M. pipe. The experiments were done on 3', 5', and 7' diameter pipe, all of the standard type with 1/2" deep corrugations, some being plain galvanized and some partly lined with asphalt to fill the corrugations. Tests were run under both full and part-full conditions.

The results were extensive and indicated values rather higher than those which had been in general use.

2.1.13. Blaisdell[13] (1960) described further model experiments on the hood inlet, as tested by Karr and Clayton in 1954. His

[illegible]

2.1.13. [illegible]

conclusions of current research. [illegible] as shown in Fig. [illegible]

2.1.14. The work of [illegible]

[illegible]

[illegible]

results more or less confirmed theirs, but he recommended the addition of an anti-vortex device. No theoretical explanation of the inlet's efficacy in causing full flow was offered.

2.2. SUMMARY OF RESULTS OF PREVIOUS RESEARCH

In this section the results and conclusions of the papers reviewed in section 2.1. are summarized, by considering separately the various factors which can influence the flow through a culvert. Taking discharge as the dependent variable, the factors determining it may be as follows:

(1) Approach conditions
(2) Headwater depth
(3) Design of inlet
(4) Slope of the culvert
(5) Size of the culvert
(6) Length of the culvert
(7) Roughness of the culvert
(8) Shape of the culvert
(9) Design of outlet
(10) Tailwater depth

2.2.1. Approach Conditions

Larson and Morris discuss the effect of approach conditions, but little experimental work seems to have been done on it. It

is probably only important at low headwater depths, when a gentle transition from the approach channel to the culvert inlet is of some benefit. If the headwater is at or above the crown of the inlet - the condition for which most culverts are designed - the headpond is virtually stagnant in most field conditions, and gradual transitions in the approach channel are of little use. The exception might be certain cases of canal culverts, but even in those cases a bellmouth or similar streamlined inlet is probably more effective than expensive wingwall construction.

2.2.2. Headwater Depth

The results of all investigations on culverts flowing part-full show that discharge is more dependent on headwater depth than on any other factor. The logical way to present both experimental results and design data is therefore in the form of rating curves. Mavis' and Schiller's rating curves (Fig. 2) are representative of the behaviour of model culverts with free outlets on steep slopes. It will be noted that there is a rapid increase in discharge up to the point at which the inlet is submerged, and that thereafter the rate of increase of discharge falls off.

On steep slopes, flow is super-critical and cannot transmit disturbances upstream. It is therefore reasonable to suppose that the discharge is controlled entirely by the inlet acting as an orifice or a weir - depending on whether or not the inlet is sub-

[illegible]

[illegible]

[illegible]

[illegible]

merged - and that the discharges for geometrically similar inlets of different sizes can be scaled by the usual Froude law, which makes discharge proportional to the 5/2 power of linear dimensions. This is the logical basis for Mavis' non-dimensional plot.

In culverts flowing full, the discharge is dependent not so much on the headwater depth as on the difference in level between headwater and tailwater. Some writers have presented model results for full flow on the same rating curve diagrams as results for inlet control operation, to demonstrate the effect of an improved inlet. This can be very misleading; full-flow data should not be shown as rating curves based on headwater depth.

The intermediate mode of culvert operation, part-full flow on mild slopes, is rather important in the case of corrugated-metal culverts. Several of the papers reviewed discuss this mode of operation, but none of the experiments described was specifically concerned with it. On mild slopes, flow is sub-critical, and friction effects are transmitted upstream, so that both the orifice effect of the inlet, and the frictional resistance of the barrel, control the discharge. Results and design data for this case are best shown as rating curves, but should not be plotted non-dimensionally.

2.2.3. Inlet Design

The general conclusion is that the inlet design may influence the discharge in two different ways, firstly by its

effect on the entrance head loss, and secondly by whether or not it causes "priming".

The entrance head loss in a pipe flowing full at the inlet is conventionally expressed as $K_eV^2/2g$, where K_e is the entrance loss coefficient and V is the velocity inside the pipe. The head loss is caused by turbulent re-expansion of the flow, which contracts to a jet in passing through the inlet. Various investigators agree fairly well on the values of K_e, and the following figures have been given for different inlet designs:

Square flush	- 0.4 to 0.5
Square projecting, C.M.pipe	- 0.8 to 0.9
Bevel flush	- no definite data, but thought to be about same as for square projecting
Hood, C.M. pipe	- 1.0
Bellmouth, r/D = 0.15	- practically zero
Concrete pipe, groove end	- 0.1

For pipes flowing part-full, there are not much data, but there are indications that K_e is less than for full flow.

A culvert will flow full if the outlet is submerged, the air trapped inside being gradually extracted by the flowing water. What is called priming, or siphoning, however, occurs with a free outlet. It is started by the flow touching the roof so as to seal it; the point of sealing then moves fairly rapidly downstream until the pipe is full. A model culvert with a square or

bevel inlet will not prime on a steep slope, except perhaps at very high headwater depths, but may do so on a mild slope, if it is not too short. On steep slopes, priming can be induced by providing a bellmouth or hood inlet, for pipe culverts, or a laterally convergent inlet for box culverts.

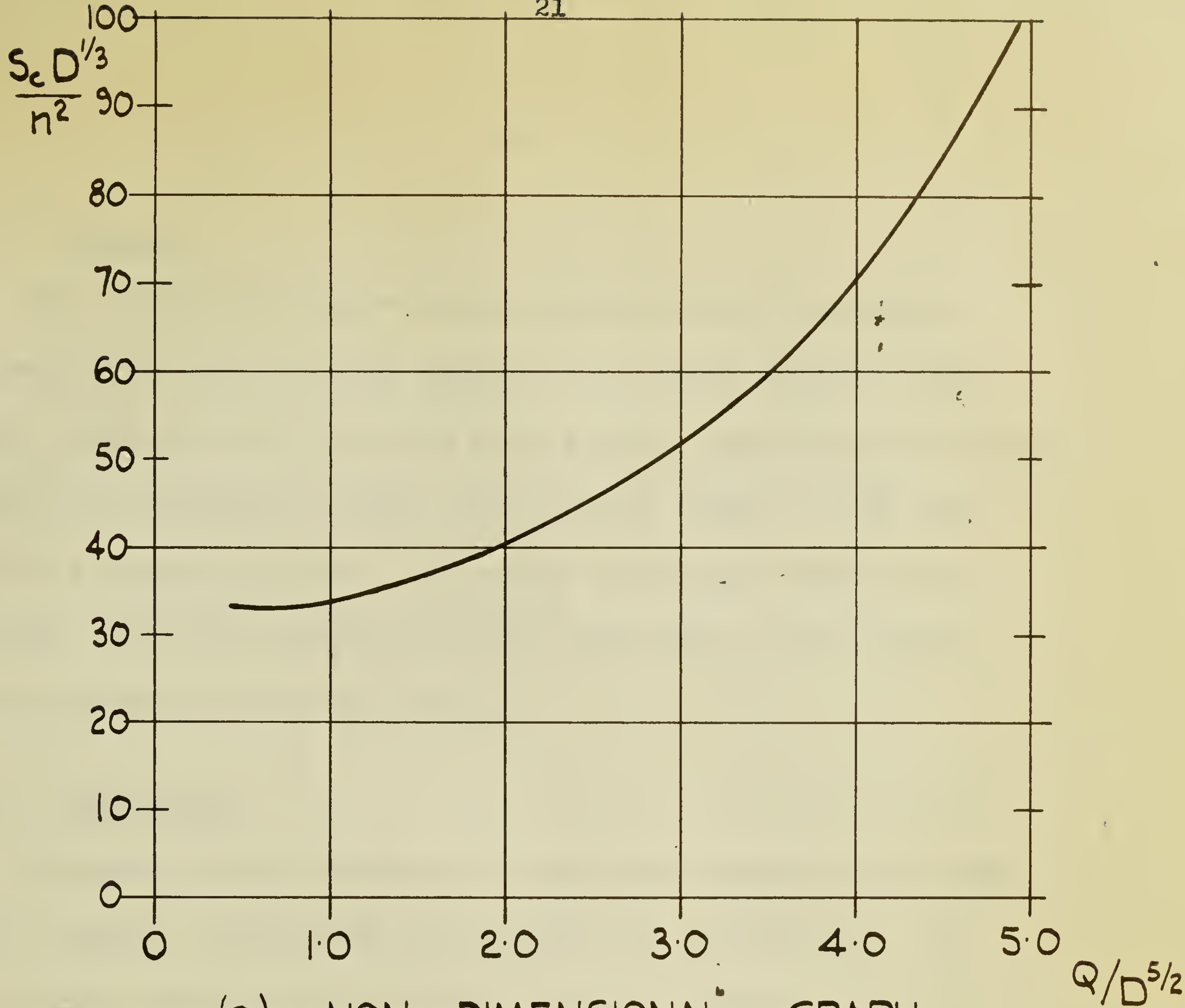
2.2.4. Slope

The effect of slope is only of direct concern in culverts flowing part-full. Mild, critical, and steep slopes were defined in Chapter 1, and their effects have been covered in sub-sections 2.2.2. and 2.2.3.

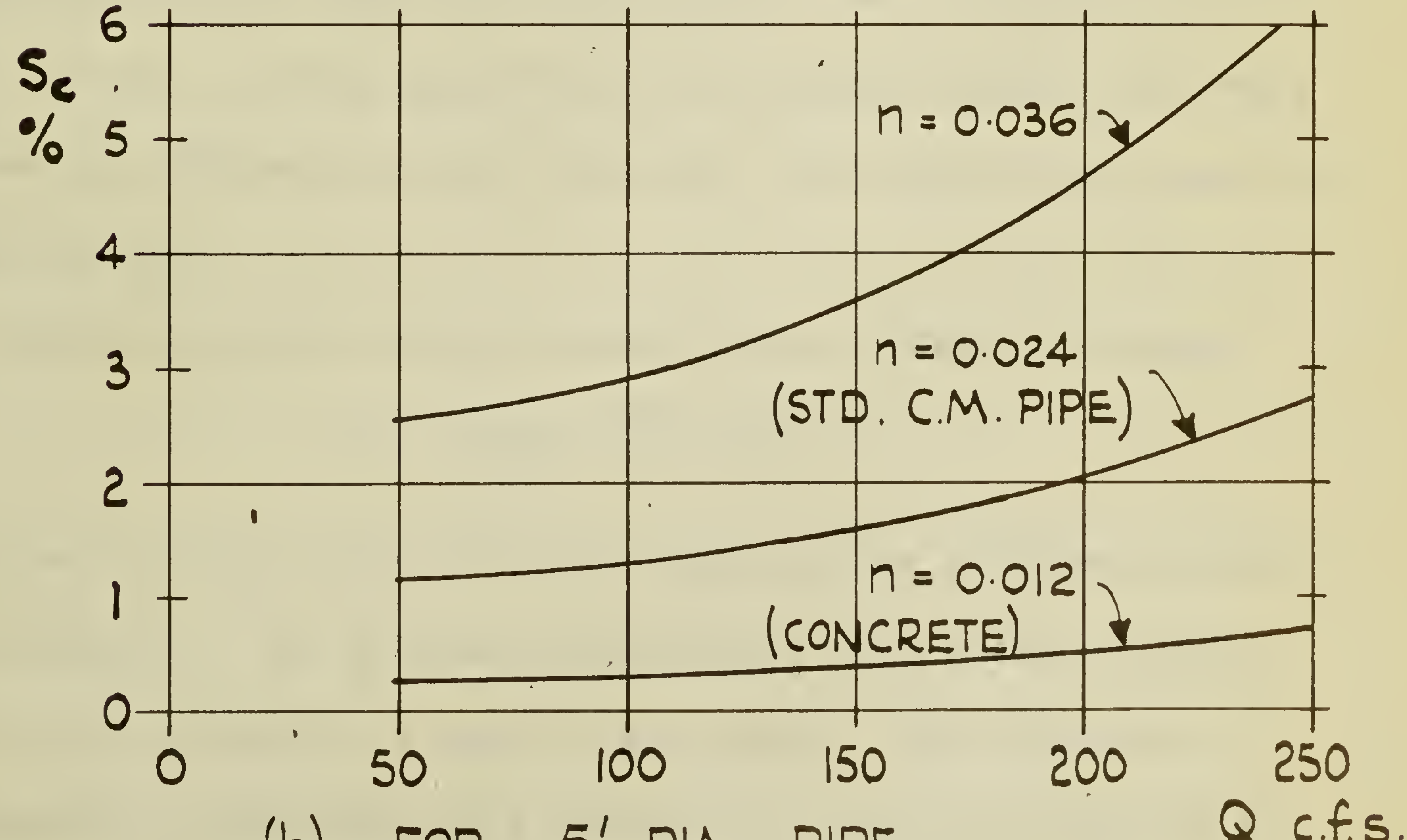
The determination of critical slope in a circular pipe is rather complicated. Fig. 3 shows a non-dimensional graph which plots $s_c D^{1/3}/n^2$ against $Q/D^{5/2}$, and also curves plotting s_c against Q for a 5' diameter pipe, for three different values of n. For small laboratory models of smooth material, s_c is of the order of 0.5%.

2.2.5. Size

If it is desired to compare the discharges, for a given headwater depth (not H/D ratio), of two culverts of the same order of size, a rough guide is that the discharge is approximately proportional to the cross-sectional area, under most flow conditions.



(a) NON - DIMENSIONAL GRAPH



(b) FOR 5′ DIA. PIPE

FIG. 3 - CRITICAL SLOPE FOR CIRCULAR PIPE

2.2.6. Length

The effect of length depends on the mode of operation. With part-full flow on steep slopes, the length makes no difference. With part-full flow on mild slopes, increasing the length reduces the discharge in most, but not all, cases. With full flow, for a fixed head drop, increasing the length reduces the discharge, but for a fixed headwater depth and a free outlet, it may increase it on steep slopes.

2.2.7. Roughness

Whether or not roughness is important depends, as in the case of length, on the mode of operation of the culvert. On steep slopes, with part-full flow, it has no effect except to determine the lower limit of steep slope, i.e., critical slope. With full flow, or with part-full flow on mild slopes, it always exercises an influence on the discharge, and can be very important in long culverts.

It has generally been assumed that the Manning formula

$$V = \frac{1.49}{n} R^{2/3} S_f^{1/2}$$

can be used to relate the friction slope to the roughness of the culvert material, and although theoretical objections could be raised, it is probably as good as any other. The roughness of the material is expressed by a value of n, which is supposed to be independent of the culvert size. If the formula is accepted,

and it is desired to use the common formula for friction head loss

$$h_f = f.\frac{L}{4R}.\frac{V^2}{2g}$$

the friction factor f may be calculated from the equation

$$f = 117\ n^2/R^{1/3}$$

which for full-pipe flow may be written

$$f = 185\ n^2/D^{1/3}.$$

There are considerable experimental data on values of n for concrete and standard C.M. pipe, and the following can be taken for design purposes:

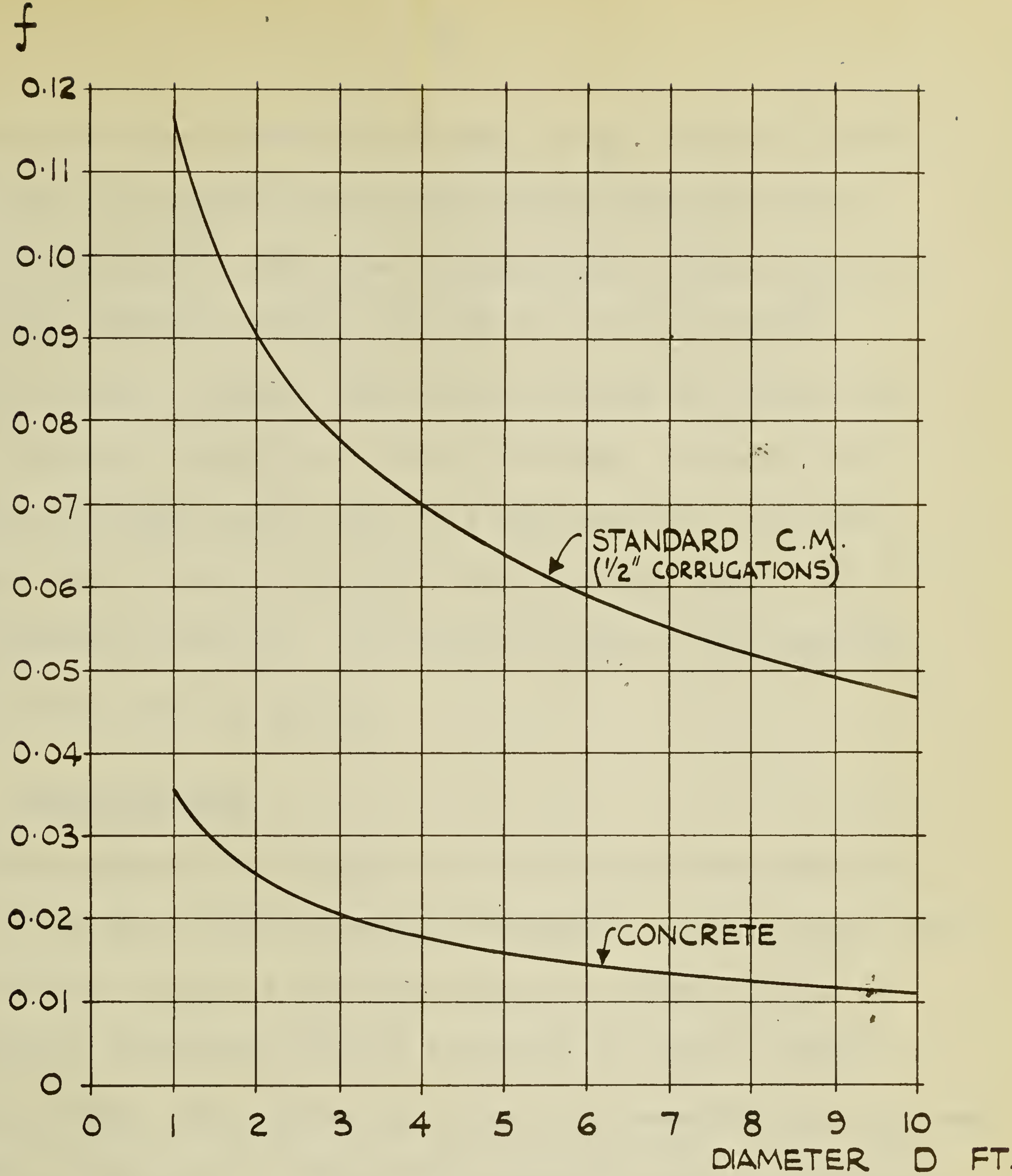
Concrete	-	0.012
Standard C.M.	-	0.024

To date there seems to have been no experimental determination for structural plate C.M. pipe. A letter from the U.S. Waterways Experiment Station indicates that model experiments have enabled values of around 0.030 to be calculated. It is doubtful if any reliance can be placed on model data for this purpose.

Fig. 4 shows graphs of f against diameter for concrete and standard C.M. pipe. The data for C.M. pipe are taken from Webster and Metcalf[12].

2.2.8. Shape

Most experimental work has been confined to circular pipe and square box shapes. Results for those seem to be interchangeable



$$h_f = f \cdot \frac{L}{D} \cdot \frac{V^2}{2g}$$

FOR PART-FULL FLOW, OR NON-CIRCULAR SHAPES, REPLACE D BY 4R.

FIG. 4 - FRICTION FACTORS FOR CULVERT PIPE

when corrected for differences in area. E.g., if experimental results for box culverts under inlet control are plotted to a base of 0.785 $Q/D^{5/2}$, they show no significant differences from results for circular culverts plotted to a base of $Q/D^{5/2}$.

In culvert design, the problem is often to secure the minimum headwater depth for a given discharge. A shape with a high width-to-depth ratio, such as a pipe-arch or a flat box, is better than a circular pipe for this purpose, since, for a given headwater elevation, the effective head at the geometric centre of the inlet is greater.

2.2.9. Outlet Design

The design of the outlet is only of importance where the culvert flows full with the outlet submerged. In this case, much of the kinetic energy of the flow, which at a plain outlet is dissipated in turbulence, can be recovered by using a flared or diverging outlet. The outlet head loss is conventionally expressed as $K_o V^2/2g$; approximate values of K_o quoted by Larson and Morris are 1.0 for a plain outlet and 0.25 for an ideal flared outlet expanding to twice the area of the barrel. The geometric design of the flaring is important. Larson and Morris recommend an included angle of not more than 12° for a box culvert outlet flaring in plan only. Probably each case should be designed individually from hydraulic principles, which will not be gone into here.

Whether or not a flared outlet is justifiable in practice depends on how large the outlet loss is, compared to other losses. For concrete culverts, it may amount to more than half the total loss, but for C.M. culverts it is seldom a very significant proportion.

2.2.10. Tailwater Depth

A free outlet was defined in section 1.4 as meaning that the tailwater was not high enough to cause a backwater effect in the barrel. With part-full flow on steep slopes, it may rise to the top of the outlet before it affects the discharge. With part-full flow on mild slopes, or with full flow, it may reduce the discharge to some extent if it rises above critical depth for the culvert. A culvert with a submerged outlet generally runs full, regardless of the inlet design or slope.

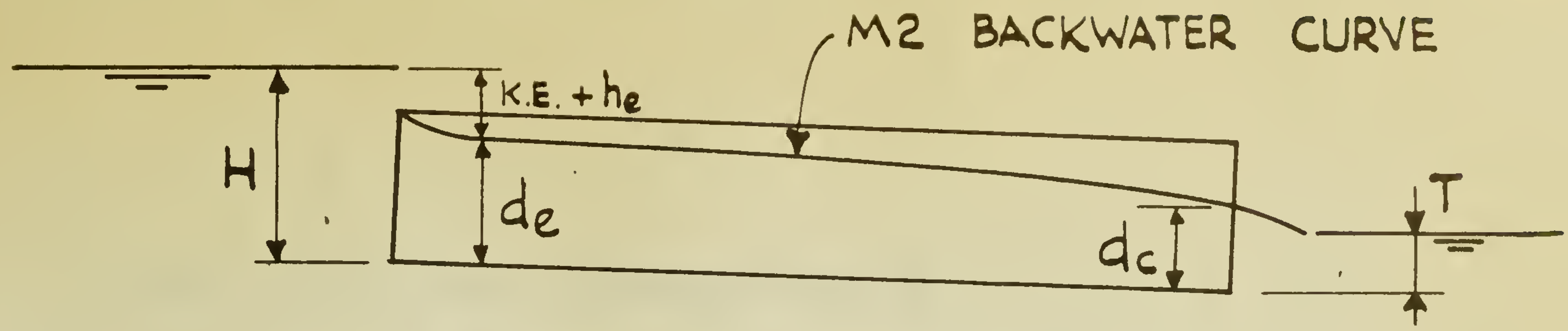
2.3. METHODS OF ANALYSIS

In this section previous research findings are examined in the light of conventional hydraulic theory.

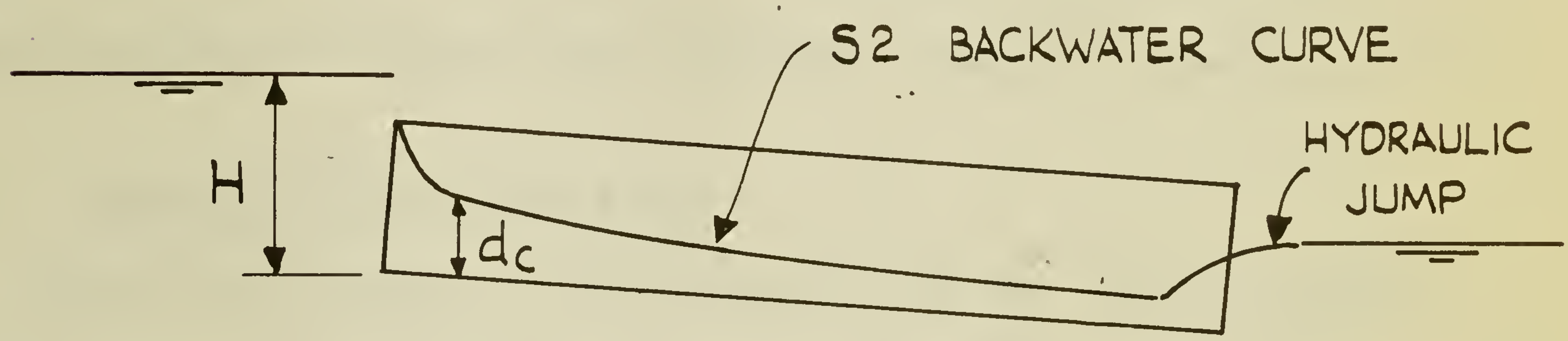
Although it is possible to classify a larger number of possible modes of culvert operation, for design purposes only four need be considered. These are illustrated in Fig. 5, and are as follows:

(i) Part-full flow, mild slope, free outlet.

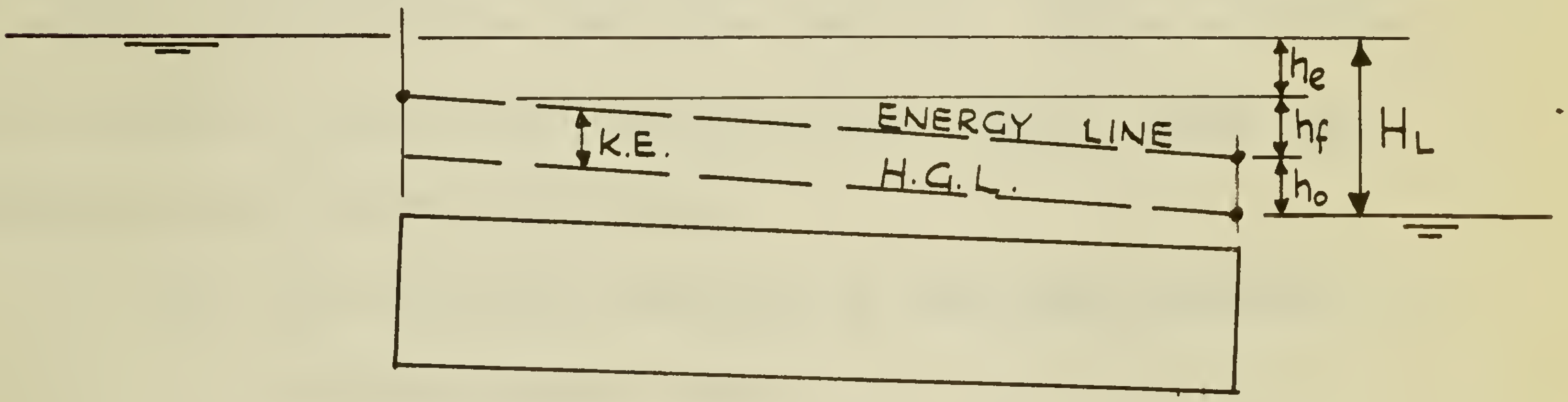
(ii) Part-full flow, steep slope.



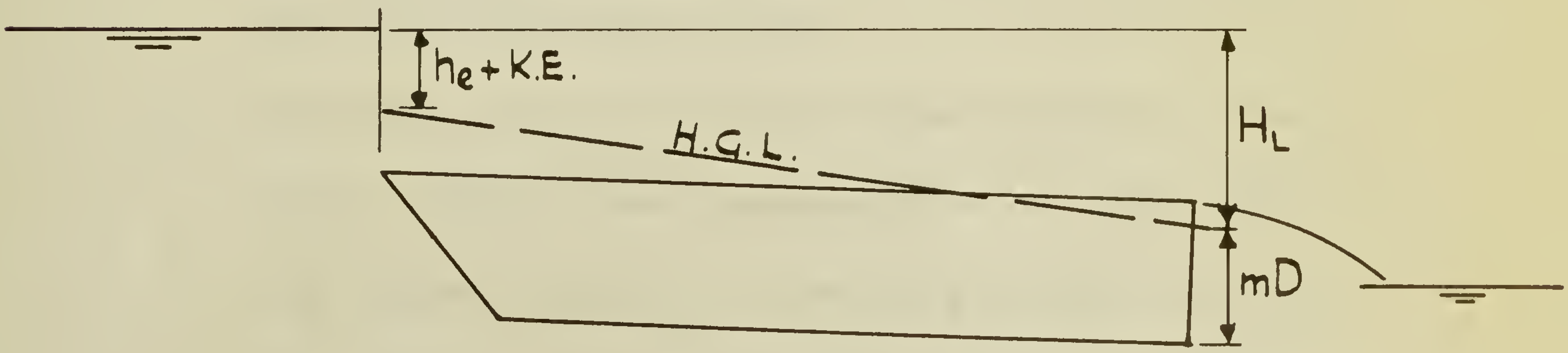
(i) PART-FULL FLOW, MILD SLOPE, FREE OUTLET



(ii) PART-FULL FLOW, STEEP SLOPE



(iii) FULL FLOW, SUBMERGED OUTLET



(iv) FULL FLOW, FREE OUTLET (PRIMED)

FIG. 5 - MODES OF CULVERT OPERATION

(iii) Full flow, submerged outlet.

(iv) Full flow, free outlet (culvert primed).

In the previous section, discharge was taken as the dependent variable. In this section, it is more convenient to regard discharge as imposed, and headwater depth as dependent.

2.3.1. Part-full Flow, Mild Slope

Flow in the barrel is subcritical, and the outlet is free, so that critical depth must occur near the outlet, which is therefore a control point. Starting from critical depth, an M2 curve, which may be calculated by the step method, defines the water surface profile up to the inlet depth d_e. There are three possible kinds of profile, depending on the length, slope, and roughness of the culvert, as follows:

(a) The M2 curve continues to the inlet without reaching normal depth.

(b) The curve reaches normal depth at some point in the barrel. Flow is then uniform between the inlet and this point.

(c) The curve hits the roof of the culvert. Previous writers have assumed that in this case the culvert primes, converting to Fig. 5 (iv). This point receives further attention in Chapter 3.

To calculate H, the velocity head and the entrance loss have to be added to d_e. The true velocity head is $\alpha V_e^2/2g$, and the entrance loss is $K_e V_e^2/2g$, so that to complete the calculation $K_e + \alpha$ must be known. Previous writers have not gone into this point very fully, but Straub, Anderson and Bowers(6) appear to have assumed that α could be taken as 1.0, and that values of K_e determined for full-pipe flow were applicable. Since they presented no experimental data for this mode of operation, there has been no way of checking the method.

(The case where the outlet is not free, i.e., T exceeds d_c, may be analyzed similarly, the backwater curve starting from T instead of from d_c.)

2.3.2. <u>Part-full Flow, Steep Slope</u>

Flow in the barrel is supercritical, so that critical depth must occur near the inlet (Fig. 5 ii). This suggests that a variation of the method outlined in 2.3.1. might be used, by adding the critical kinetic energy and the entrance loss, if any, to d_c to give H. Straub, Anderson and Bowers(6) followed roughly this method and claimed good agreement with Mavis' data up to H/D = 1.3. At higher heads, the calculated discharges become increasingly larger than Mavis'(2); since the experimental data are extensive and consistent, the method must be rejected as inapplicable where H/D exceeds 1.3. Plate 1, which shows the behaviour of a 3-1/2"

To calculate θ, [illegible] head and the momentum flux [illegible] $V_1^2/2g$ [illegible]

and the momentum flux [illegible] $V_1^2/2g$ [illegible]

[illegible] θ_1 [illegible]

[illegible]

[illegible]

values of [illegible]

[illegible]

[illegible]

This means [illegible]

[illegible]

[illegible]

4.1.4 [illegible]

Thus in the [illegible]

[illegible]

variation of [illegible]

[illegible]

[illegible]

this method and obtained good agreement with their data up to [illegible]

[illegible]

larger than [illegible]

and [illegible]

[illegible] Figure [illegible] which shows the behaviour of [illegible]

(a) H/D = 1.0

(b) H/D = 2.0

PLATE 1 - INLET CONDITIONS IN SMOOTH MODEL CULVERT ON STEEP SLOPE

diameter model, suggests the reason why: in the first photograph, at H/D = 1.0, the flow pattern is not unlike the idealized profile of Fig. 5 (ii), whereas in the second, at H/D = 2.0, there is a complex three-dimensional pattern, and the usual calculation for critical conditions, assuming hydrostatic pressure distribution, cannot apply.

If Mavis' data are analyzed according to the orifice formula

$$Q = c_d A \sqrt{2gh}$$

and h is taken as the net head to the centre of the inlet $(H-\frac{D}{2})$, the graph of c_d against H/D shown in Fig. 6 results. Also shown is a graph of c_d for a circular thin-plate orifice, based on data from Schoder and Dawson's "Hydraulics"[14] (1934). At H/D over 2.0, the agreement is very close. Straub, Anderson and Bowers, by introducing another empirical factor into the orifice equation, claimed agreement down to H/D = 1.4.

To sum up, it appears that part-full flow on steep slopes may be analyzed by critical flow theory for low heads, and orifice theory for high heads, if appropriate experimentally determined coefficients are inserted in the equations. For practical purposes there is no point in going through this procedure, and data are best taken directly from experimental non-dimensional rating curves such as Mavis' and Schiller's.

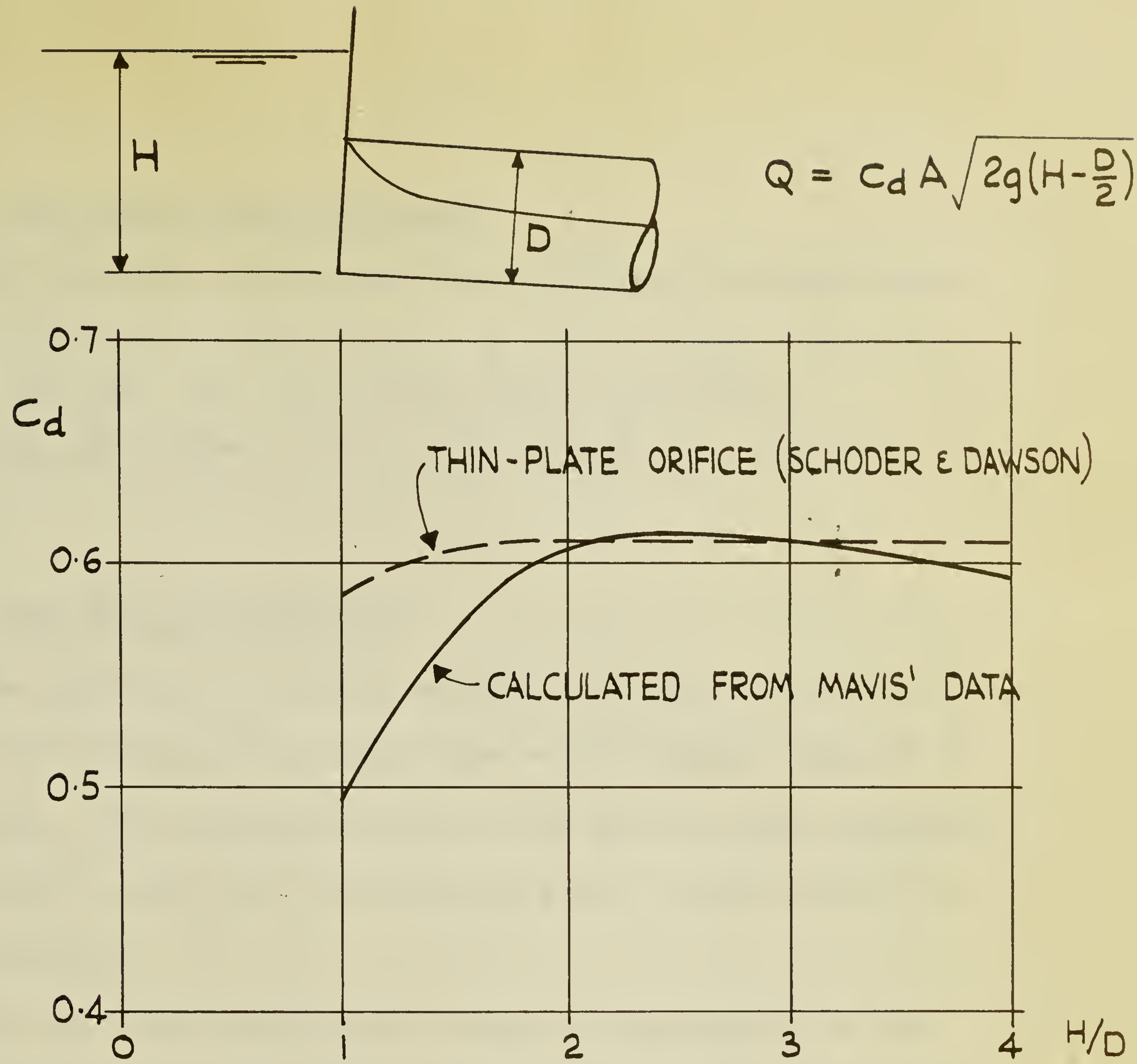


FIG. 6 - MAVIS' DATA ANALYSED BY ORIFICE THEORY

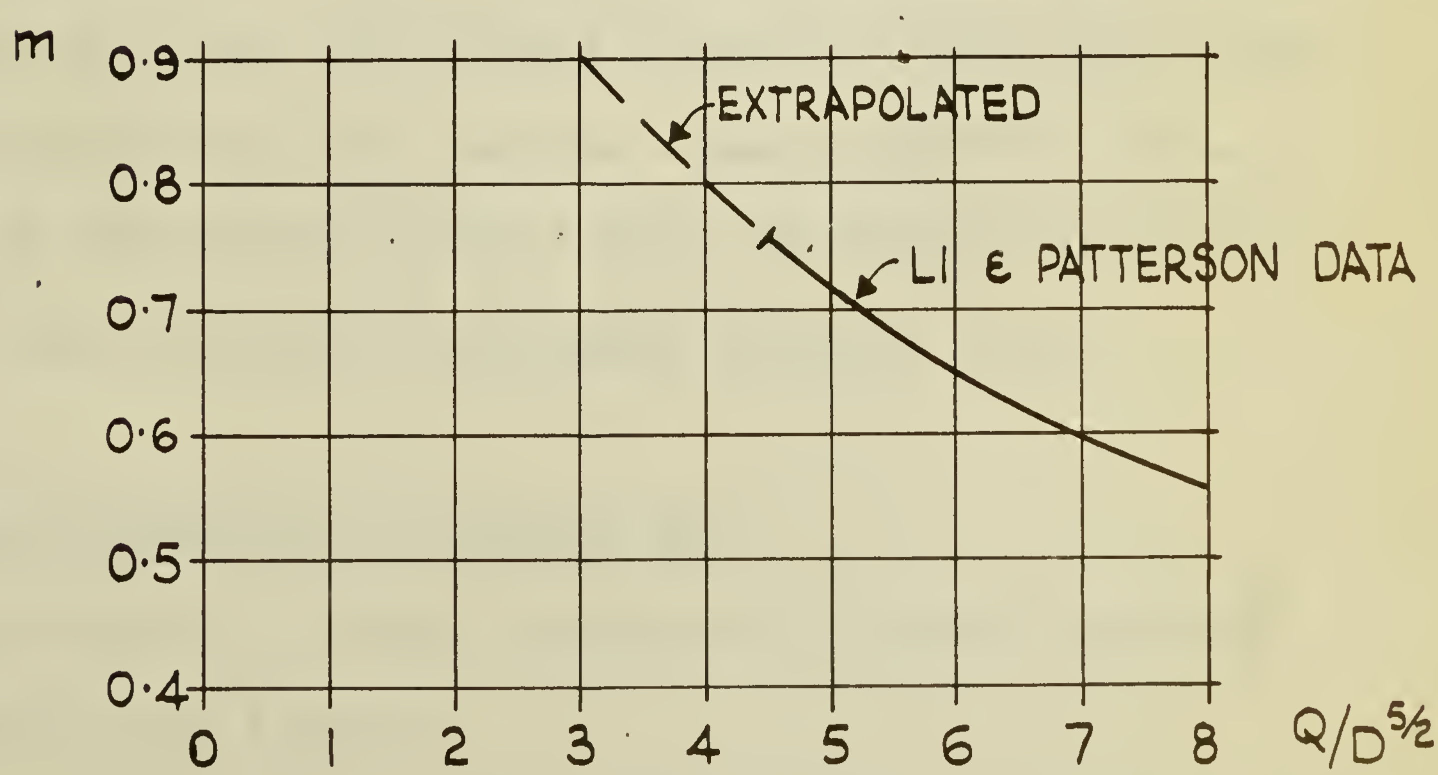


FIG. 7 - HYDRAULIC GRADE LINE AT FREE OUTLET
(SEE FIG. 5 iv)

2.3.3. Full Flow, Submerged Outlet

This is the standard pipe-flow condition, with the equation

$$H_L = h_e + h_f + h_o = (K_e + fL/D + K_o)\ V^2/2g$$

Values of K_e were given in 2.2.3., of f in Fig. 4, and of K_o in 2.2.9.

2.3.4. Full Flow, Free Outlet

The equation is the same as above, but H_L is the drop from headwater to the hydraulic grade line at the outlet, instead of to tailwater. The downstream part of the pipe is under negative pressure head at the roof, so that the H.G.L. at the outlet is below the crown.

The only experimental data found on the location of the H.G.L. at the outlet was that of Li and Patterson(8), obtained from small pipe models. They expressed the height of the H.G.L. above the invert as mD, and plotted m against $V/\sqrt{gD}$. Fig. 7 was obtained by converting their data to a base of $Q/D^{5/2}$, for ease of comparison with rating curves. Since the data is so scanty, in case of doubt it would be advisable to take m = 0.9.

2.4. DESIGN INFORMATION IN CURRENT USE

Three sources of design information currently available are reviewed in this section.

2.4.1. Armco Handbook

Data for pipe culverts with free outlets are presented on slope-capacity charts, one of which is reproduced in simplified form on Fig. 8 (a). This chart is for $H/D = 1.0$ and for $n = 0.021$, but charts are also given for H/D up to about 1.5, and for $n = 0.015$.

The maximum discharges shown are higher than Mavis' and similar experimental data. At $H/D = 1.0$ the difference is about 25%, and at higher heads it is greater. The maximum discharges were calculated by critical flow theory, assuming $\alpha = 1.0$ and $K_e = 0$.

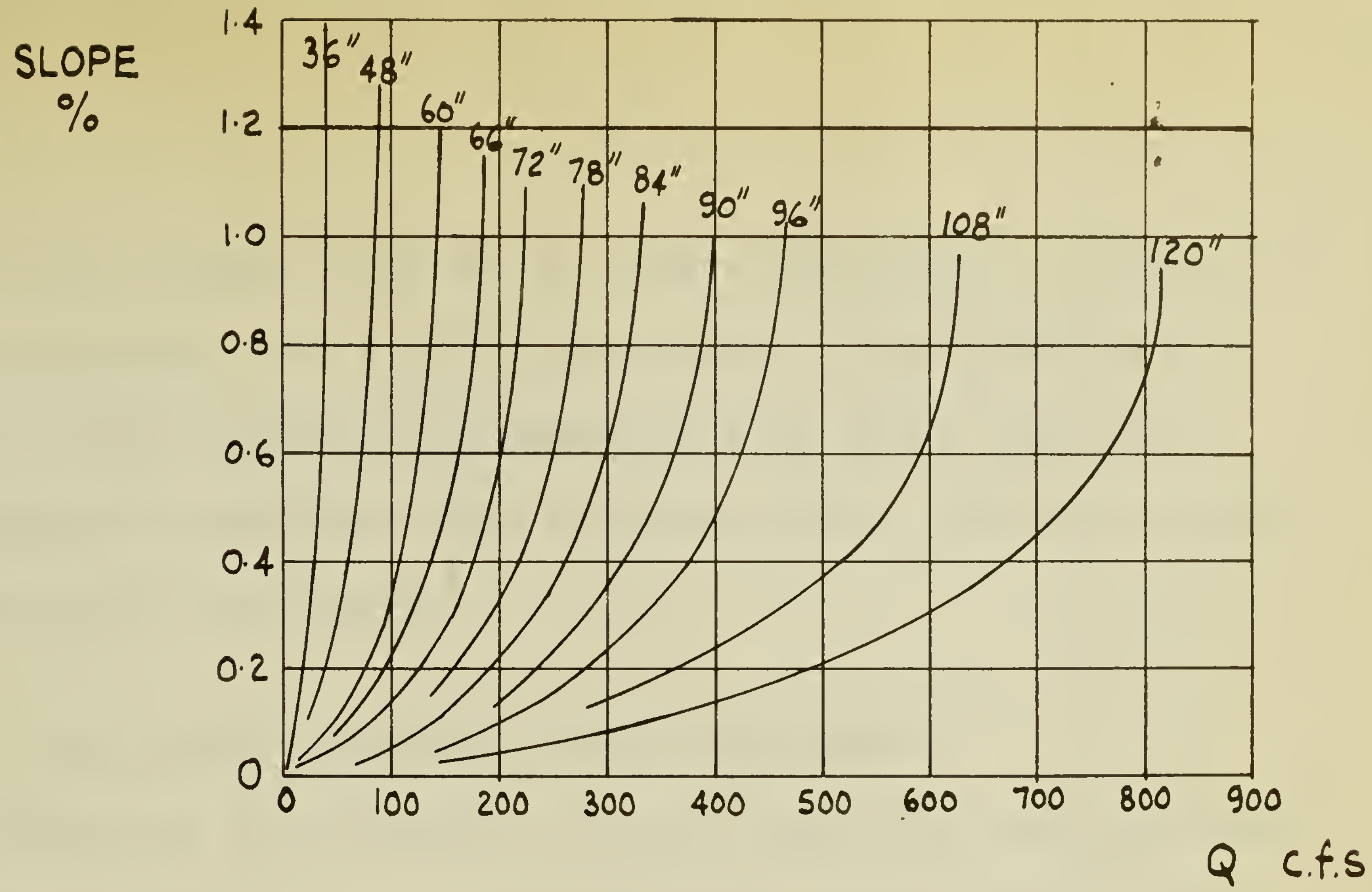
Since the value of $n = 0.021$ is slightly low for standard C.M. pipe, and much too low for structural plate, the indicated critical slopes are too small.

The discharges shown for very flat slopes are too small, and would only apply to very long culverts. E.g., the charts for $H/D = 1.0$ show zero discharge at zero slope. Culvert length is not mentioned.

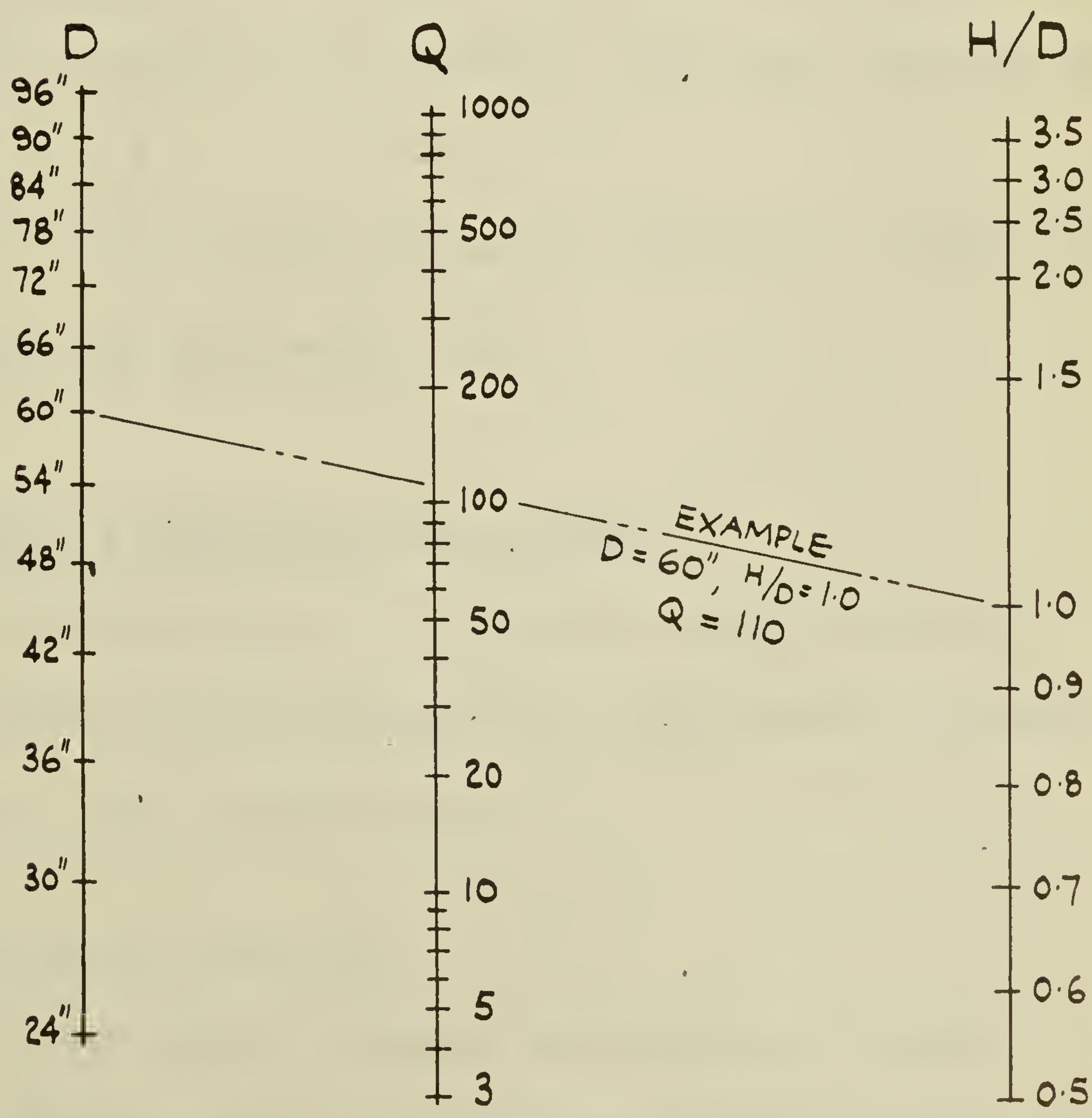
No data are given for pipe-arches. Users generally assume that figures for a pipe of the same area apply.

2.5.2. Concrete Pipe Handbook

A miscellany of information from various research papers is presented. The writers have, however, shown such zeal in claiming the superiority of concrete over C.M.



(a) ARMCO CHART (H/D = 1·0)



(b) SCHILLER'S NOMOGRAPH (SQUARE PROJ. INLET)

FIG. 8 - PUBLISHED DESIGN DATA FOR C.M. CULVERTS

pipe that the result is often to confuse the reader. Schiller's design nomographs (see 2.1.10.) are given; a simplified copy of the one for C.M. pipe is reproduced in Fig. 8 (b). The nomographs apply to part-full flow on steep slopes, and were derived from Schiller's model data.

2.4.3. U.S. Bureau of Public Roads Nomographs

These are in two series, one for full flow, and the other for inlet control, i.e., part-full flow on steep slopes. The chart for standard C.M. pipe in the first series is reproduced in modified form in Fig. 9; the others are for C.M. pipe-arches, concrete pipe, and concrete box culverts. They are based on the equation given in 2.3.3.

The charts in the second series are similar to Schiller's, and mainly based on the same model data.

2.5. DEFICIENCIES IN PREVIOUS PUBLICATIONS

Some of the deficiencies in previous research and in published design data, which prompted the experiments described in the next chapter, are listed below.

2.5.1. Deficiencies in Research

(a) Except for Yarnell, Nagler and Woodward's work in 1926, and Webster and Metcalf's on friction factors in standard

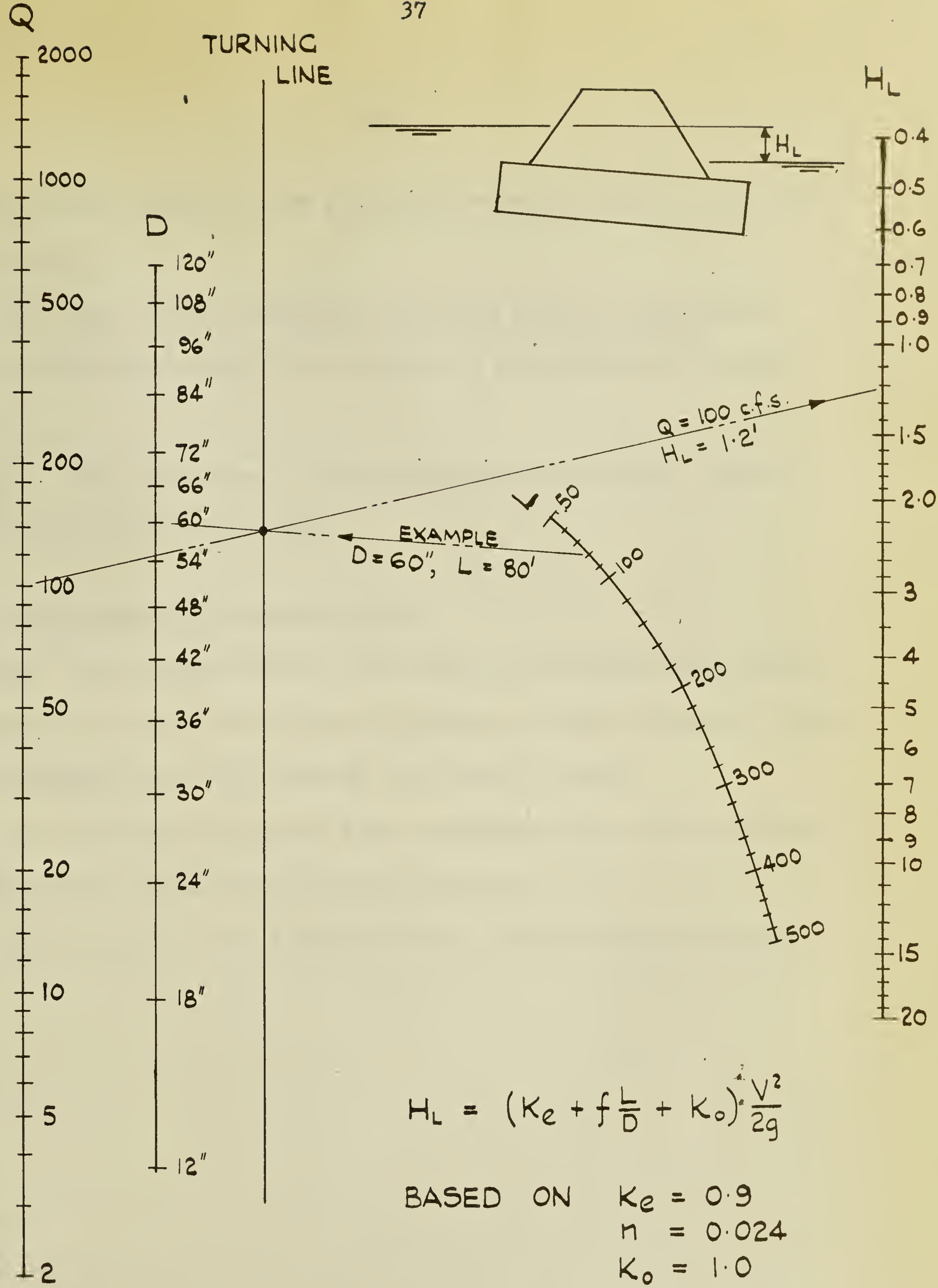


FIG. 9 - B.P.R. NOMOGRAPH FOR STD. C.M. PIPES, SQUARE PROJECTING INLET, FULL FLOW

C.M. pipe (1959), none of the published work has dealt with full-size culverts.

(b) Very little attention has been given to part-full flow on mild slopes, which is believed to be important for C.M. culverts.

(c) There has been no determination of friction factors for structural plate C.M. pipe.

2.5.2. <u>Deficiencies in Design Data</u>

(d) The Armco charts do not agree with model data where they should, and are based to some extent on faulty theory. They make no allowance for differences in culvert length.

(e) The Schiller and B.P.R. nomographs for inlet control do not give any guidance on critical slopes.

(f) No charts are available for culverts on mild slopes.

CHAPTER 3 - ALBERTA FIELD EXPERIMENTS, 1960

3.1. EXPERIMENTAL INSTALLATION

The site is on the Cairnhill Spillway Channel of the Western Irrigation District, 8 miles S. and 3-1/2 miles W. of Strathmore, Alberta. The channel is a natural depression; at its head, 2-1/2 miles upstream of the site, a side spillway gate on the main W.I.D. canal enables excess water to be diverted down the channel.

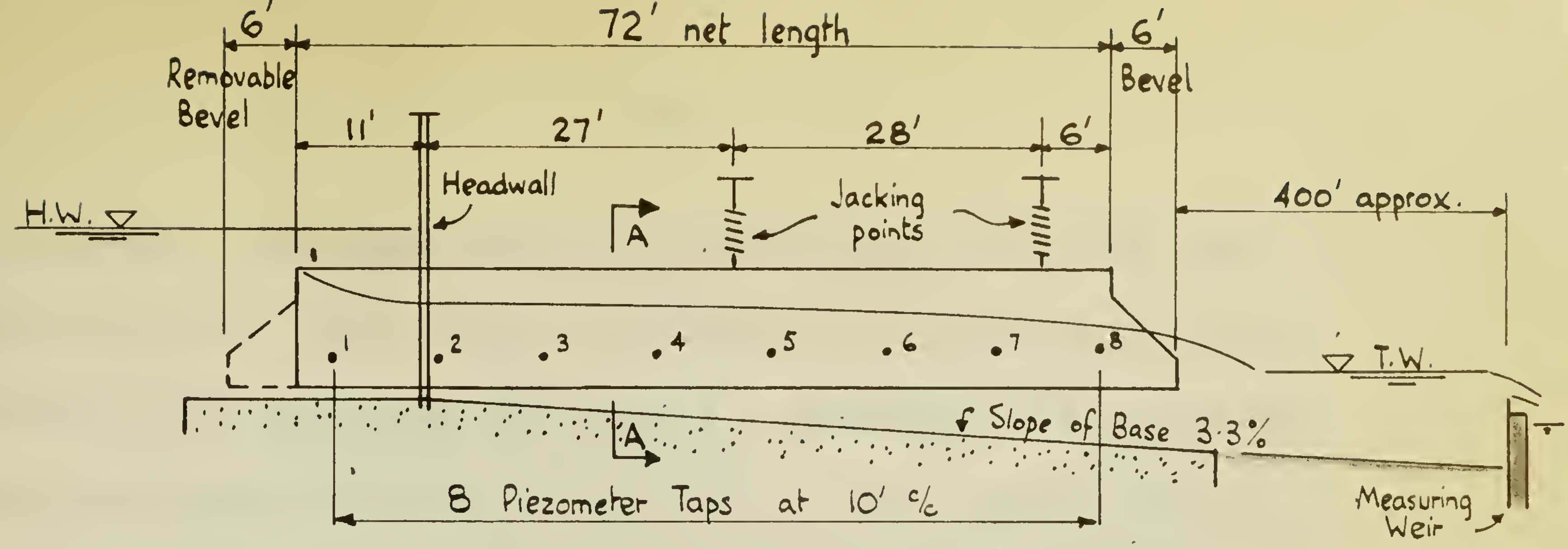
At the site, the channel passes under a municipal road. An old culvert, which had proved too small in past floods, was removed, and replaced by a timber bridge, and the necessary headgates and supports for the experimental culvert were incorporated in the structure. A general view of the site, and a view of the bridge and culvert during construction, are shown on Plate 2.

The culvert was nominally 60" diameter, of structural plate corrugated-metal, with a net length of 72'. Details of the culvert and measuring weir are shown in Fig. 10. The culvert plates were bolted together on the site and no attempt was made to seal the joints, these being made as in a normal highway

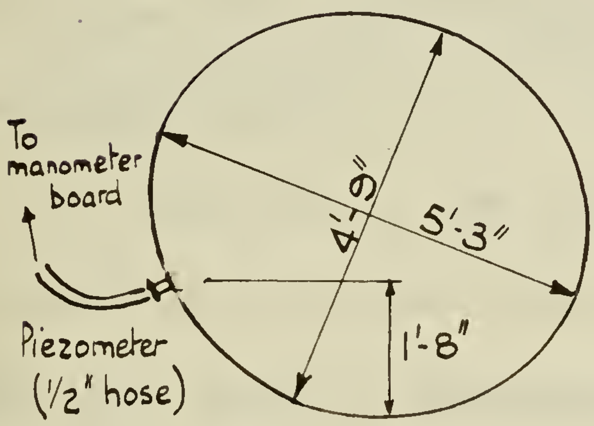
(a) EXPERIMENTAL SITE FROM DOWNSTREAM
Measuring Weir in Foreground

(b) STRUCTURE DURING CONSTRUCTION
From Downstream

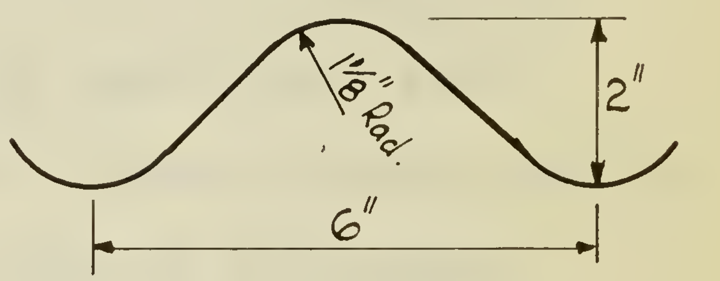
PLATE 2 - EXPERIMENTAL CULVERT - GENERAL VIEWS



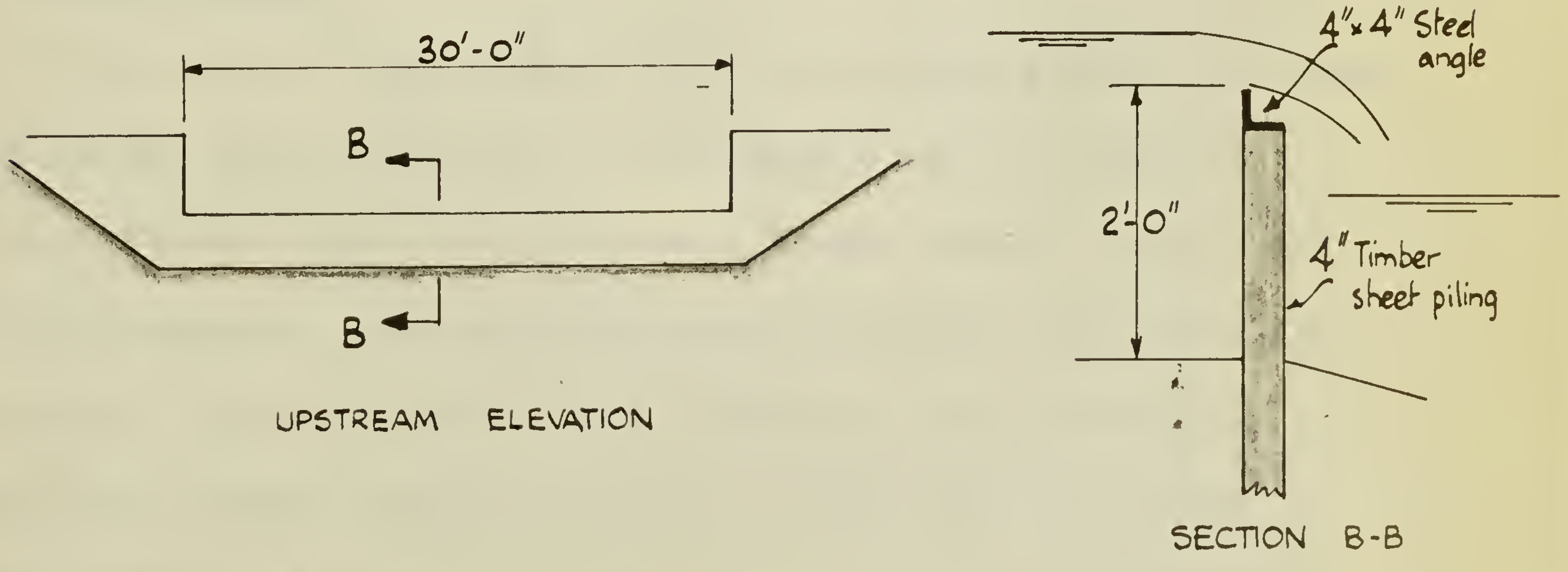
SKETCH PROFILE OF CULVERT



CROSS-SECTION A-A
Dimensions to inside corrugations



DETAIL OF CORRUGATIONS



DETAILS OF MEASURING WEIR

FIG. 10 - DETAILS OF EXPERIMENTAL CULVERT

installation. The slope could be varied from 0 to 3.3%, and various types of inlet could be fitted: the bevel projecting and hood inlets are shown on Plate 3. Headwater and tailwater gauge boards were attached to the ends of the culvert, and a slope pointer, reading on a fixed scale, was attached to the outlet. The 1/4" diameter piezometer taps every 10', which were located in the troughs of the corrugations, were connected by 1/2" hose to a bank of perspex manometer tubes fixed to a common scale board.

In addition to the readings at the experimental site, some readings were also taken on two 84" diameter culverts located on the same channel, approximately 1/2 mile and 1-1/2 miles downstream of the site.

3.2. TEST PROGRAM

The original aim of the 1960 test program was to run tests at discharges up to 250 c.f.s., on slopes of 0, 1, 2 and 3%, with 4 different types of inlet-bevel flush, square projecting, hood and bellmouth. Due to construction problems the installation was not completed until early September, and operating difficulties further delayed testing so that only two weeks' testing proved possible before the irrigation system closed for the season. The requirements of the irrigation users limited the maximum draw-off to about 150 c.f.s., which was only available

(a) BEVEL PROJECTING INLET

(b) HOOD INLET

PLATE 3 - EXPERIMENTAL CULVERT - INLETS

for limited periods. Thus it was not possible to run the full range of tests, and the program had to be revised to obtain the most useful data from the facilities available.

3.3. TEST PROCEDURE

The test observations were recorded as shown on the upper parts of the combined data and analysis sheets included in the Appendix. Tests numbered F1, F2, etc. refer to the 60" culvert, the G series refer to the first 84", and the H series to the second 84". The readings are described below.

3.3.1. Observed Depths

Headwater and tailwater depths H and T were read on gauge boards, and are measured from the invert levels at the inlet and outlet ends of the 72' net length. The H/D ratio for the 60" culvert was calculated by using the average diameter of 5'.

In some of the later tests, an "inlet depth d_e" was measured at a point 8' downstream of the inlet, through a hole in the culvert roof. The flow at this point was generally free of disturbance caused by the inlet, and the reading was used mainly as a check on the piezometers.

In most of the tests, the outlet depth d_o was measured at the end of the 72' length, and used as a check on the piezometers.

3.3.2. <u>Discharge Measurements</u>

The weir head h was read on a gauge board in a stilling well. The discharge was calculated from the formula

$$Q = 3.33 \times 30 \times (h + V_a^2/2g)^{3/2}$$

where V_a = velocity of approach, estimated from the discharge and cross-section of the channel.

The weir crest was formed of a structural steel angle, not specially sharpened. It therefore had a small radius, about 1 mm. Data in Schoder and Dawson's "Hydraulics" (1934) indicate a plus correction of about 1% for the applicable range of heads. This correction, and the small minus correction for end contractions, were ignored, as being small compared with possible errors in observing h. It is believed that the error in the recorded discharges is certainly less than 5%, and probably less than 3%.

In 4 of the earlier tests (F6, F7, G2, H2), the tailwater of the weir rose above the crest. This condition was quickly corrected by deepening the channel, and correction factors were applied to the discharge formula for those tests, according to Table 12 of the Bureau of Reclamation's "Water Measurement Manual". Since correction factors for the submerged crest condition are not too reliable, the discharges for those tests are slightly suspect.

The $Q/D^{5/2}$ ratio for the 60" culvert was calculated by taking D = 5'.

§ 7.1. [illegible]

The small head h was read on a [illegible] of a stilling well. The discharge was calculated from the formula:

[illegible]

where K_s = [illegible] correction, [illegible] from the [illegible] of the channel.

The weir [illegible] [illegible] [illegible] [illegible] [illegible] contained [illegible] [illegible] [illegible] [illegible] [illegible] a glass correction of about [illegible] for the [illegible] range of heads. This correction, and the small [illegible] correction for end contractions, were ignored, as being small compared with probable errors in measuring h. It is believed that the error in the [illegible] discharge is certainly less than [illegible] and probably less than [illegible].

In [illegible] of the earlier runs ([illegible]) the [illegible] of the weir rose above the crest. This condition was [illegible] by [illegible] the [illegible], and corrections [illegible] applied to the discharge [illegible], according to Table 12 of the Bureau of Reclamation's "Water Measurement Manual". Since [illegible] the [illegible] [illegible] not too reliable, the [illegible] for these [illegible] are slightly [illegible].

The [illegible] for the [illegible] was calculated [illegible] [illegible].

Table 1, copied from Larson and Morris[3] (1948), gives various hydraulic factors for part-full flow in circular pipes. To calculate the critical depth d_c, the depth ratio d/D corresponding to the recorded value of $Q/D^{5/2}$ was read from the table, and multiplied by the actual vertical dimension of the pipe (4.8' for the 60" culvert) to give d_c.

3.3.3. Piezometer Readings

The zero of the piezometer gauge board was 0.05' below the fixed invert elevation of the culvert at No. 2 piezometer, i.e., at the headwall (see Fig. 10).

It is believed that in the first 10 tests, insufficient care was taken to clear the piezometer tubes of air before each set of readings, and that the readings are therefore suspect. The readings on No. 1 piezometer were obviously affected by curvature of flow, and were generally discounted. The readings on No. 8, while they might have been expected to be similarly affected, generally agreed fairly well with direct depth measurements at the outlet.

At the higher discharges, the manometers were subject to fluctuations of up to about 0.3', and mean readings had to be estimated visually.

The possibility has to be admitted that the piezometers did not read true static heads, due to the influence of the

[illegible]

Table [illegible] (19[illegible]). [illegible] [illegible].

To calculate [illegible] [illegible] [illegible] [illegible] and multiplied by the [illegible] [illegible].

[illegible]

3.3.3. [illegible]

The zero of the piezometer gauge board [illegible] the fixed survey elevation of [illegible] piezometer [illegible] (Fig. [illegible]).

It is believed [illegible] [illegible].

[illegible]

At the higher [illegible] [illegible].

[illegible]

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{V_c^2}{2gD}$	$\frac{S_c D^{1/3}}{n^2}$
0.01	0.0013	0.0065	0.0006	0.00331	80.5
0.02	0.0037	0.0130	0.0025	0.00709	68.9
0.03	0.0069	0.0198	0.0055	0.00990	53.6
0.04	0.0105	0.0261	0.0098	0.0135	50.5
0.05	0.0147	0.0326	0.0153	0.0168	48.1
0.06	0.0192	0.0388	0.0220	0.0205	45.0
0.07	0.0242	0.0457	0.0298	0.0235	42.5
0.08	0.0294	0.0512	0.0389	0.0272	41.1
0.09	0.0350	0.0575	0.0491	0.0305	39.9
0.10	0.0409	0.0636	0.0605	0.0340	38.9
0.12	0.0534	0.0754	0.0868	0.0411	37.4
0.14	0.0668	0.0871	0.1176	0.0481	36.0
0.16	0.0811	0.0985	0.1530	0.0554	35.2
0.18	0.0961	0.110	0.1928	0.0626	34.5
0.20	0.1118	0.121	0.237	0.0699	34.0
0.22	0.1281	0.131	0.286	0.0774	33.7
0.24	0.1449	0.141	0.339	0.0851	33.5
0.26	0.1623	0.152	0.396	0.0926	33.3
0.28	0.1800	0.162	0.457	0.1002	33.1
0.30	0.1982	0.171	0.523	0.1082	33.1
0.32	0.217	0.180	0.592	0.1158	33.2
0.34	0.236	0.189	0.666	0.1244	33.3
0.36	0.255	0.198	0.743	0.1325	33.5
0.38	0.274	0.206	0.825	0.1408	33.7
0.40	0.293	0.214	0.910	0.1502	34.0
0.42	0.313	0.222	1.000	0.1580	34.2
0.44	0.333	0.230	1.093	0.1681	34.7
0.46	0.353	0.237	1.190	0.1768	35.1
0.48	0.373	0.244	1.291	0.1860	35.7
0.50	0.393	0.250	1.396	0.1960	36.2

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{V_c^2}{2gD}$	$\frac{S_c D^{1/3}}{n^3}$
0.52	0.413	0.256	1.50	0.207	36.9
0.54	0.433	0.262	1.62	0.218	37.6
0.56	0.453	0.268	1.73	0.229	38.4
0.58	0.472	0.273	1.85	0.239	39.3
0.60	0.492	0.278	1.98	0.250	40.1
0.62	0.512	0.282	2.11	0.262	41.4
0.64	0.531	0.286	2.24	0.277	42.8
0.66	0.550	0.290	2.38	0.290	44.0
0.68	0.569	0.293	2.52	0.305	45.4
0.70	0.587	0.296	2.67	0.320	47.4
0.72	0.605	0.298	2.82	0.336	49.2
0.74	0.623	0.300	2.98	0.356	51.4
0.76	0.641	0.302	3.15	0.375	53.6
0.78	0.657	0.303	3.32	0.396	56.4
0.80	0.674	0.304	3.51	0.422	59.8
0.82	0.689	0.304	3.70	0.450	63.8
0.84	0.704	0.304	3.91	0.479	68.3
0.86	0.719	0.303	4.15	0.515	73.8
0.88	0.732	0.301	4.41	0.564	81.5
0.90	0.744	0.298	4.70	0.625	91.0
0.91	0.750	0.296	4.87	0.655	96.6
0.92	0.756	0.293	5.06	0.697	104.0
0.93	0.761	0.292	5.27	0.743	111.9
0.94	0.766	0.289	5.52	0.808	123.0
0.95	0.771	0.286	5.82	0.886	136.2
0.96	0.775	0.283	6.18	0.986	154.8
0.97	0.779	0.279	6.67	1.142	183.1
0.98	0.782	0.272	7.41	1.398	232.0
0.99	0.784	0.267	8.83	1.971	367.0
1.00	0.785	0.250	--	--	--

TABLE 1 - HYDRAULIC FUNCTIONS FOR PART-FULL FLOW IN CIRCULAR PIPES

[illegible]

corrugations on the flow in their vicinity. However, in those tests where depths 8' downstream of the inlet and at the outlet were measured directly, there was reasonable agreement with the depths calculated from the piezometer readings.

3.4. GENERAL DESCRIPTION OF TESTS

Notes on the general behaviour of the culverts, as observed during the tests, are included in this section.

3.4.1. Tests F1 to F4.

The culvert was on the maximum slope of 3.3%, and the discharge was increased from 25 to 67 c.f.s. Plate 4 shows photographs of test F3. The projecting bevel inlet seemed to behave like a 3-sided weir. Flow through the culvert appeared to be at nearly uniform depth, with an aerated surface and a pattern of small waves moving downstream.

Due to the relative levels of the culvert and the measuring weir, the weir caused a backwater effect at the culvert outlet when the culvert was on a 3% slope, except at high discharges.

3.4.2. Tests F5 to F7

The slope was changed to 2%, and the discharge run up to 118 c.f.s., submerging the inlet. The transition from unsubmerged to submerged inlet conditions was quite smooth. The flow through

(a) INLET - BEVEL PROJECTING

(b) OUTLET

PLATE 4 - TEST F3 - SLOPE 3.3 % , Q = 56 c.f.s.

the culvert became highly aerated on the surface. Plate 5 illustrates test F6.

3.4.3. <u>Tests F8 to F10</u>

The slope reverted to 3.3%, and the discharge was run from 106 to 153 c.f.s. In Plate 6, showing test F8, the rough water surface and the pattern of V-shaped waves can be distinguished.

3.4.4. <u>Tests F13 to F17</u>

The culvert was raised to 1% slope, and the discharge run from 50 to 148 c.f.s. Falling backwater curves (i.e. drawdown curves) were noted through the culvert. Up to 100 c.f.s., the flow was not noticeably aerated. After the inlet was submerged, at about 100 c.f.s., a continuous vortex occurred immediately above the crown of the inlet. Stopping the vortex by floating a board on the surface did not appear to affect the flow through the culvert. In tests F16 and F17, at over 140 c.f.s., the pipe appeared to run full for a short distance downstream of the inlet.

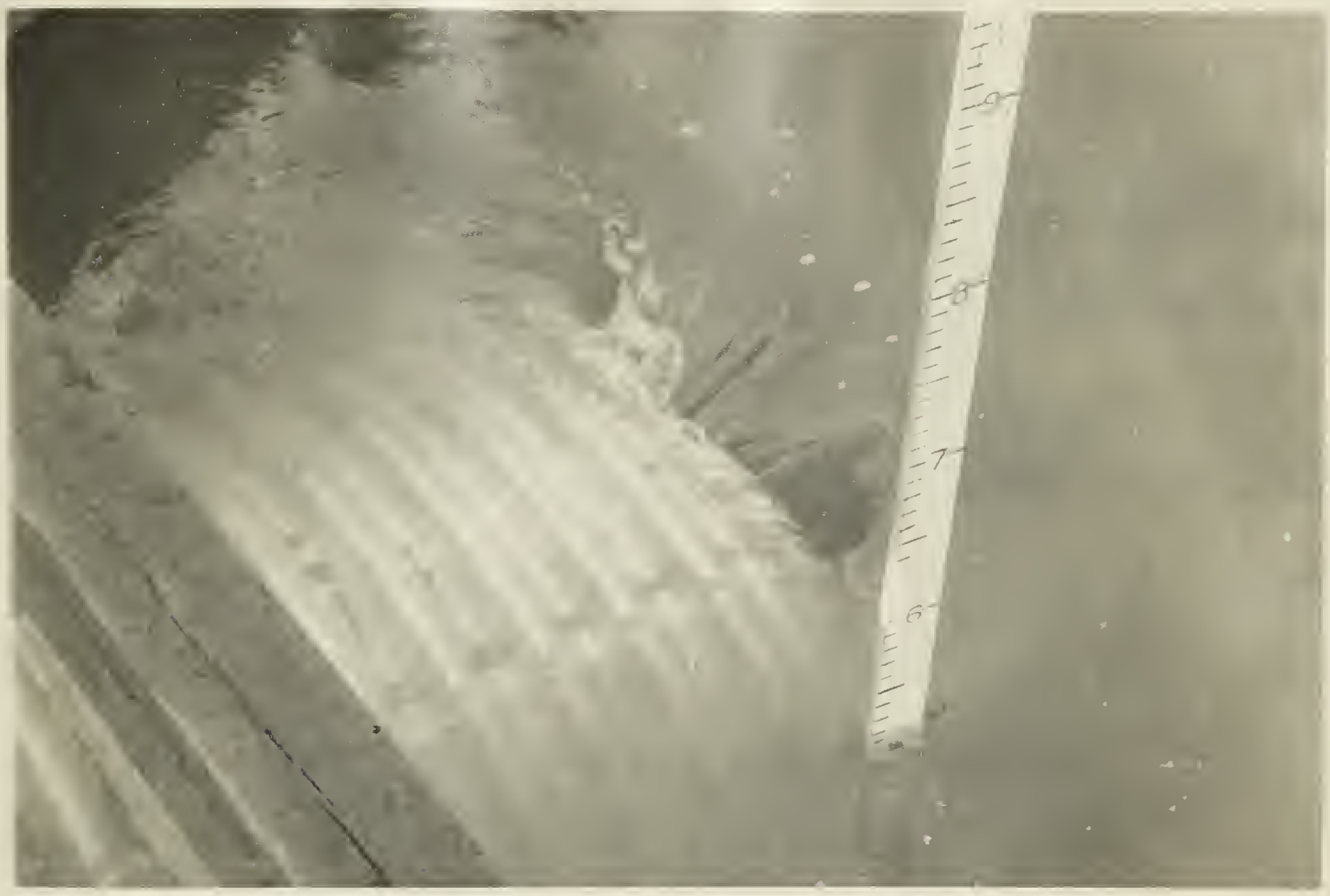
3.4.5. <u>Tests F18 to F21</u>

The bevel end was removed, to leave a square projecting inlet, and the discharge was run from 48 to 123 c.f.s. The flow pattern at the inlet was noticeably different and much smoother. A vortex occurred with the inlet submerged.

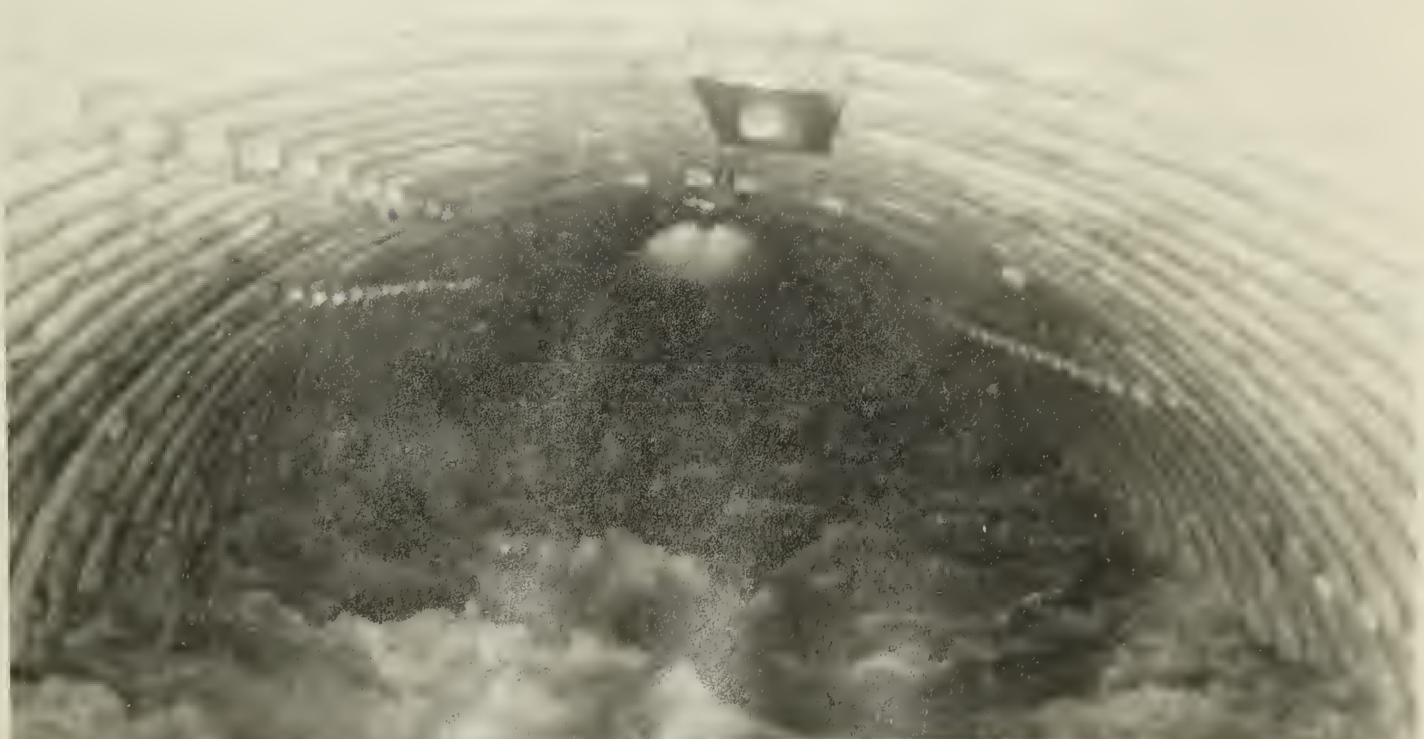
(a) INLET - PROJECTING BEVEL

(b) OUTLET

PLATE 5 - TEST F6 - SLOPE 2% , Q = 96 c.f.s.



(a) INLET - BEVEL PROJECTING
Gauge Board shows depth in feet above inlet invert



(b) LOOKING UPSTREAM FROM OUTLET

PLATE 6 - TEST F 8 - SLOPE 3.3 % , Q = 106 c.f.s.

3.4.6. Tests F22 to F26

The slope was changed to zero, mainly to find whether the culvert would prime (as models do on zero slope). The discharge was run from 48 to 148 c.f.s. Inlet conditions for tests F22 and F23 are shown on Plate 7. Above about 130 c.f.s., the inlet vortex became weak and intermittent. Although part of the pipe ran full from 118 c.f.s. upwards, it did not prime. As in all the tests, the bolted joints leaked water quite freely. The flow did not seem as highly aerated as on the higher slopes.

3.4.7. Tests F27 to F31

The bevel inlet plates were fixed upside down, to form a hood inlet (see Plate 3b), the slope was changed to 1%, and the discharge was run from 59 to 157 c.f.s. Inlet conditions were very smooth, and no vortex occurred. Again there was no evidence of priming, although part of the pipe ran full at discharges above 125 c.f.s. Plate 8 illustrates test F31.

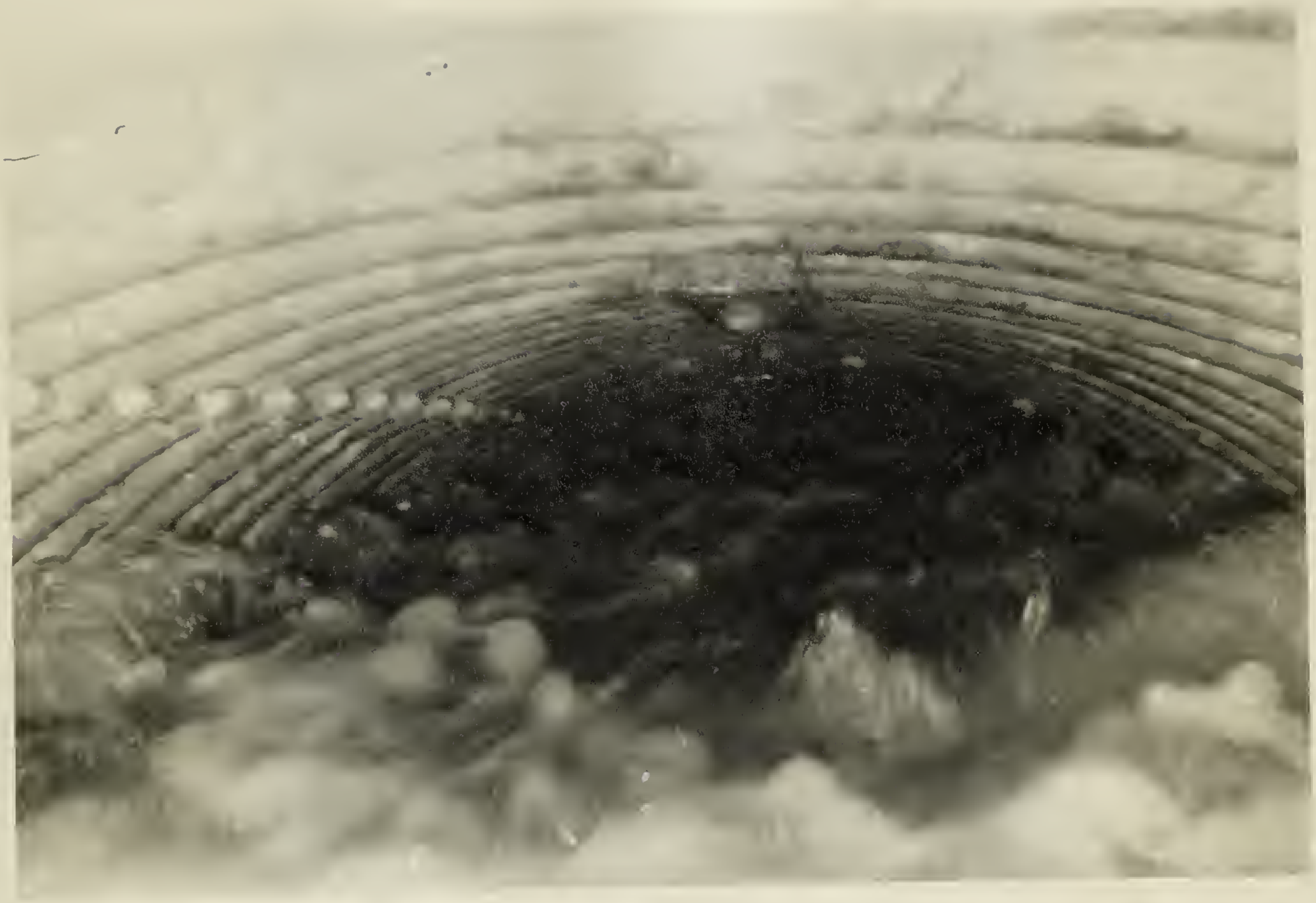
3.4.8. Tests F32 to F36

The bevel plates were re-fixed normally, and plywood sheets attached, to form a bevel flush inlet. The discharge was run from 30 to 111 c.f.s., but no differences from previous tests without the plywood were noticed. Plate 9 shows test F35; the contrast with the inlet photograph on Plate 7(b) should be noted.

(a) TEST F22 - SLOPE 0 , Q = 48 c.f.s.

(b) TEST F23 - SLOPE 0 , Q = 76 c.f.s.

PLATE 7 - FLOW CONDITIONS AT SQUARE PROJECTING INLET



(a) LOOKING UPSTREAM FROM OUTLET

(b) OUTLET

PLATE 8 - TEST F31 - SLOPE 1% , Q = 157 c.f.s., HOOD INLET

(a) INLET - BEVEL FLUSH

(b) OUTLET

PLATE 9 - TEST F35 - SLOPE 2.8% , Q = 78 c.f.s.

3.4.9. Tests G1 to G4, H1 and H2

Insufficient water was available to submerge the inlets of the 84" culverts. Conditions of flow appeared generally similar to those in the 60" culvert. Plate 10 illustrates test G3.

3.5. ANALYSIS OF RESULTS

In the analysis of experimental results, especially where, as in this case, the data are somewhat limited, it is easy to be misled into drawing wrong conclusions. This part of the report is therefore treated in considerable detail. The aim of the analysis is to arrive at a rational basis for applying the results to culverts of different diameters and different lengths, and, if possible, with higher headwater depth ratios. It must be remembered that the experimental culvert was relatively short, and that the maximum H/D ratio reached in the tests was 1.7.

3.5.1. Comparison with Design Data

The first stage in analysis was to plot H against Q for all the tests on the 60" culvert, as shown on Fig. 11, without regard to differences in slopes and inlets, and to compare the results with the design data discussed in section 2.4. The line plotted on Fig. 11 from data in the Armco handbook gives discharges considerably higher than the experiments; at the usual design

(a) INLET

(b) OUTLET

PLATE 10 - TEST G3 - 84" CULVERT , Q = 136 c.f.s.

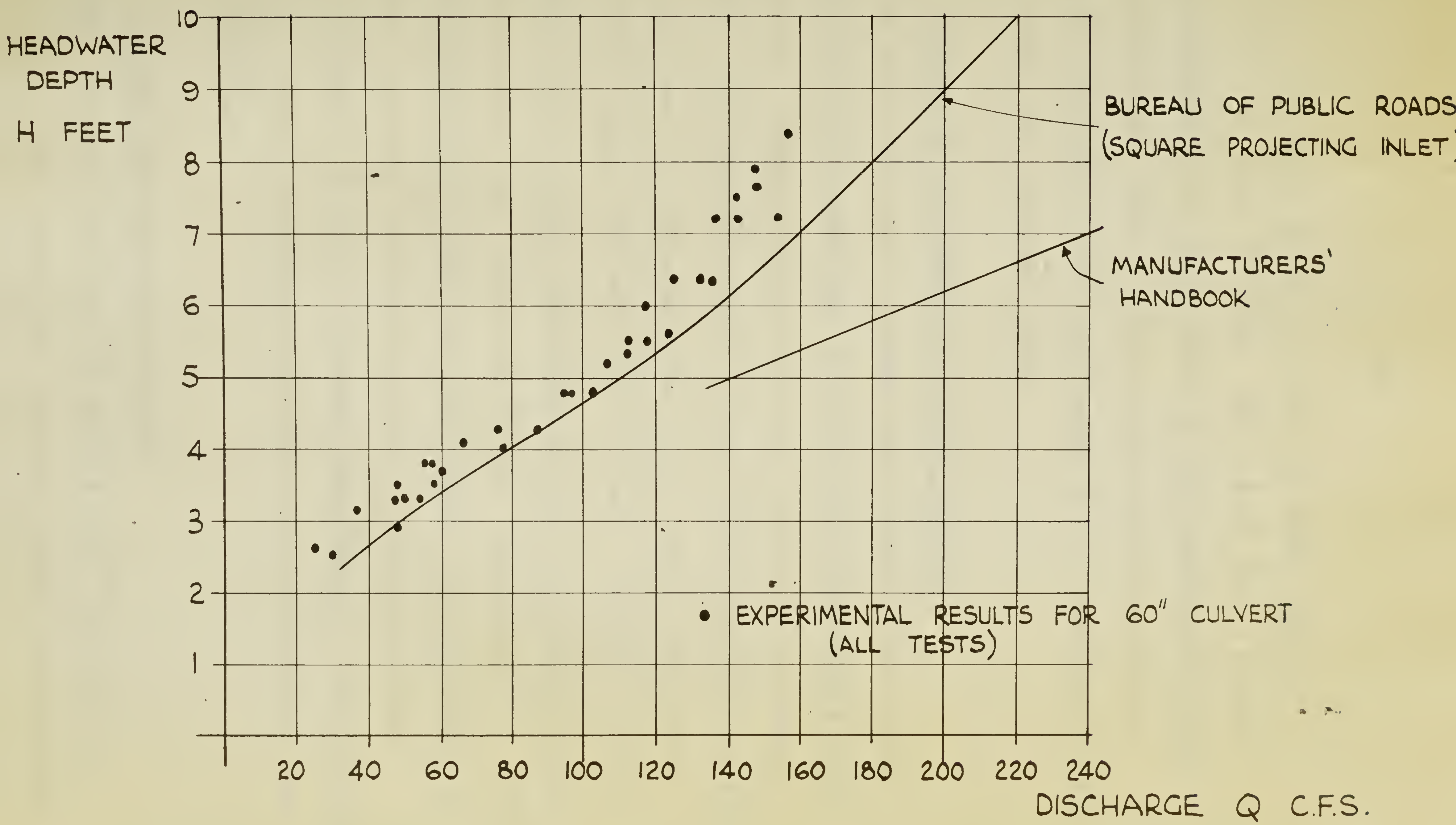


FIG. 11 – EXPERIMENTAL RESULTS COMPARED WITH DESIGN DATA FOR 60" C.M. CULVERT

depth of 5', the Armco figure is about 25% high, but at a depth of 7' it is about 60% high. The B.P.R. curve, plotted from the nomographs, is quite close to the experimental points, and the differences might at first sight appear negligible, although at the higher heads there seems to be a tendency for the points to veer away from the curve.

3.5.2. Comparison with Schiller's Model Results

The next stage was to compare the relevant field results with Schiller's non-dimensional curve obtained from models (Fig. 2, Chapter 2). Only certain of the field results were directly comparable, since Schiller did not test a bevel projecting or hood inlet, and since his curves were for steep slopes, which ruled out the field results for zero slope at least. On Fig. 12, all the results for the square projecting and the bevel flush inlets, at slopes of 1% or over, have been plotted, together with a curve representing Schiller's data for these inlets. The agreement is remarkably close, and is thought to give a fairly good check on the accuracy of the field discharge measurements. Unfortunately, none of the tests at discharges over 125 c.f.s. was suitable for direct comparison.

3.5.3. General Non-dimensional Diagram

Fig. 13 shows a plot of H/D against $Q/D^{5/2}$ for all the tests on both the 60" and the 84" culverts, classified according

[illegible]

[illegible] and the difference [illegible] appears negligible, although at the highest [illegible] seems to be a tendency for the [illegible]

1.5.6. [illegible]

The [illegible] Whillock's [illegible]

1.5.7. [illegible]

Fig. 13 [illegible]

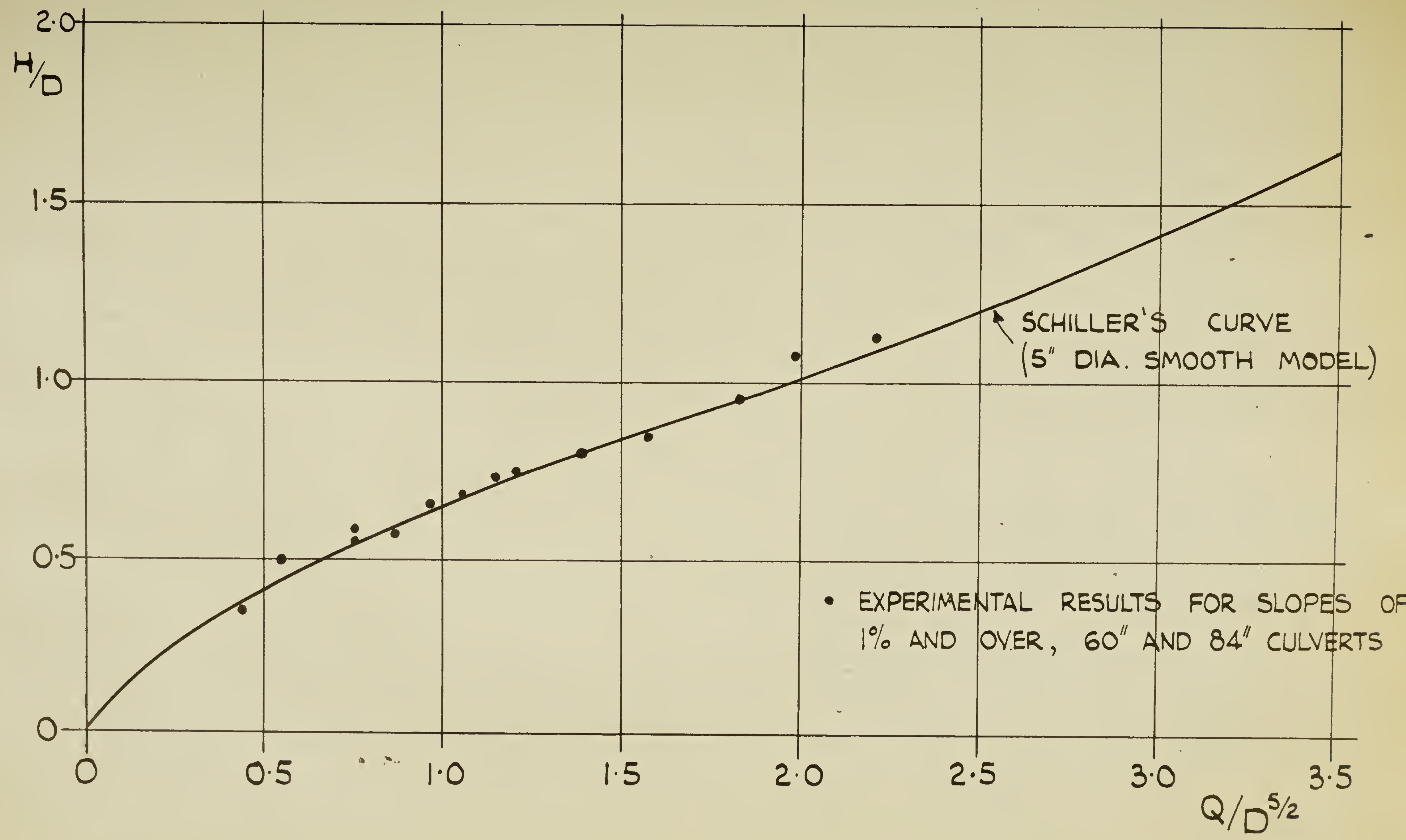


FIG. 12 - EXPERIMENTAL RESULTS COMPARED WITH SCHILLER'S MODEL RESULTS - SQUARE PROJECTING OR BEVEL FLUSH INLET (SELECTED RESULTS ONLY - SEE 3.5.2)

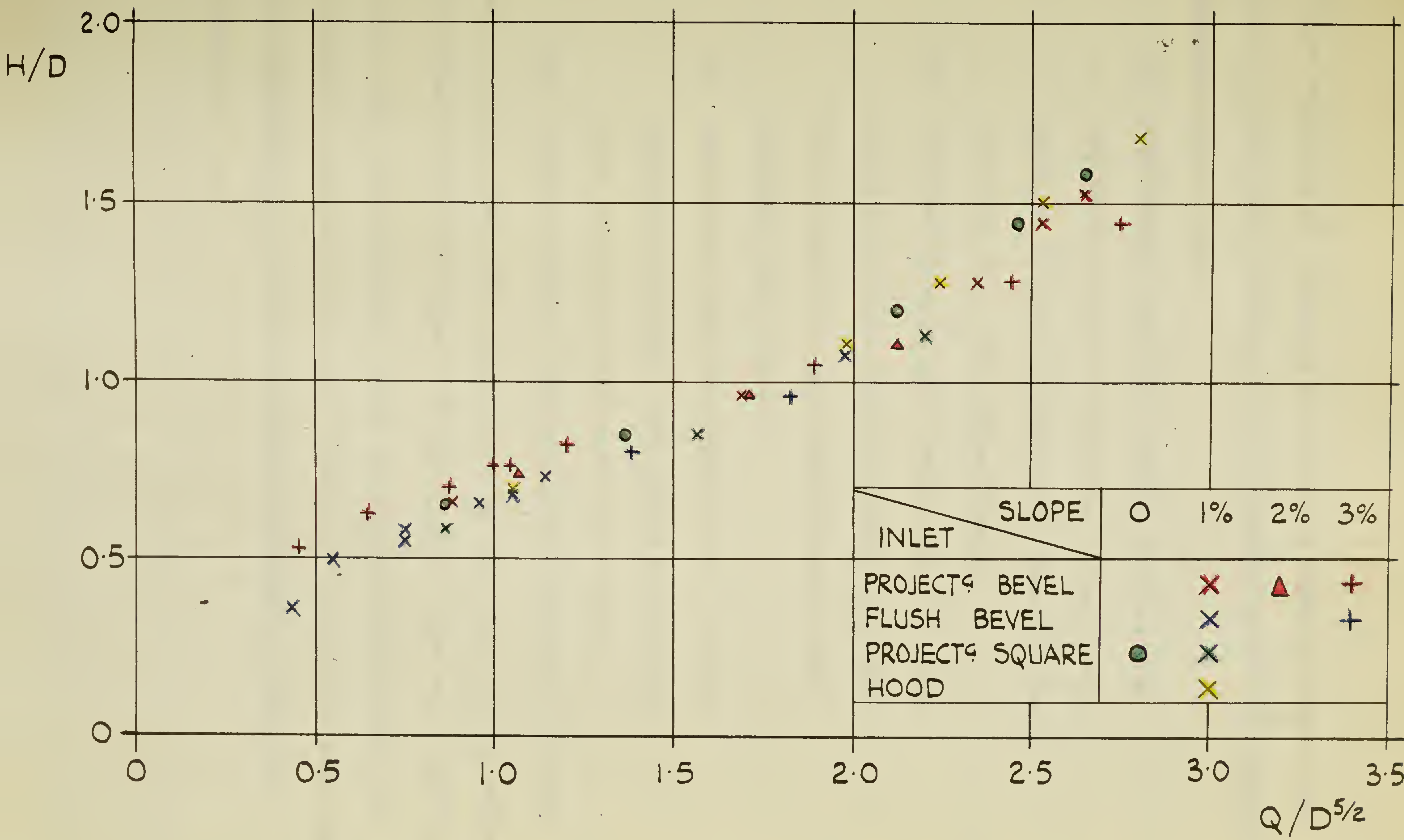


FIG. 13 - NON-DIMENSIONAL PLOT OF ALL EXPERIMENTAL RESULTS

to slope and type of inlet. It is not too easy to detect the effects of different factors on this diagram, but the following tendencies can be noted:

(1) Considering only the results for the bevel projecting inlet (red points), the 1% slope appears to give significantly less discharge than the 3% at the higher heads.

(2) Considering only the results for the square projecting inlet (green points), the zero slope appears to give significantly less discharge than the 1%.

(3) Considering only the results for 1% slope (diagonal crosses), the square projecting inlet appears to give the best results.

Fig. 13 thus seems to indicate that differences in both slope and inlet shape were significant in the field tests, although their effects were not large. The fact that slope was significant indicates that the headwater depth discharge relationship was determined to some extent by pipe friction, so that the next step in the analysis was to try to determine Manning's roughness coefficient.

3.5.4. Calculation of Roughness Coefficient

In the combined data and analysis sheets in the Appendix, alternative tables are given for calculating n, either by means of a step backwater calculation, or by a uniform flow calculation. The uniform flow calculation, which was used wherever the piezometer readings and visual observation indicated no significant departure from uniform depth, is a straight application of Manning's formula, and requires no explanation.

The backwater calculation, which was used wherever the evidence indicated a significant departure from uniform flow, is basically the standard step method as given in many hydraulics texts (e.g., Streeter's "Fluid Mechanics",[15] 1958, Sec. 76), and will not be explained in detail here. It should be noted, however, that the velocity head was corrected by introducing a K.E. correction factor α , estimated as 1.3. This value was calculated from the value of n found in the uniform-flow tests, using Streeter's[16] empirical formula

$$\alpha = 1 + 2.5 f \qquad (f = 185 n^2/D^{1/3})$$

The calculations are not very sensitive to variations in α , so that only an approximate figure is required. It is recognized that the method of tabulating the calculations is not strictly correct, in that some of the columns should be averaged between increments of d, but one or two trial calculations by the more exact method indicated that the small difference in results

did not justify the extra labour. The general method was to go through the calculation, using trial values of n, until the calculated water surface profile agreed closely with the test profile as indicated by the piezometers; only the final calculation is shown on the sheets.

In calculating values of A and R, allowance was made for the slightly flattened shape of the 60" culvert as follows. The depth was divided by the actual vertical dimension of the culvert (4.8'), to give the depth ratio d/D. Corresponding values of A/D^2 and R/D were then read from Table 1, and multiplied by 25.0 and 5.0 respectively.

Table 2 shows the calculated values of n. They range from 0.029 to 0.039, with an overall average of 0.036. There is no indication of any systematic pattern in the variations, and, for lack of evidence to the contrary, the scatter must be ascribed to observational and computational inaccuracy. The values calculated by the backwater method do seem to be significantly higher than the rest, which probably reflects an inherent bias in the method, due in part to the approximate method of tabulation discussed above. The values are examined further in section 3.6.

3.5.5. <u>Entrance Drawdown Calculation</u>

In 2.3.1. it was seen that, for culverts flowing part-full on mild slopes, the difference between the headwater depth and the

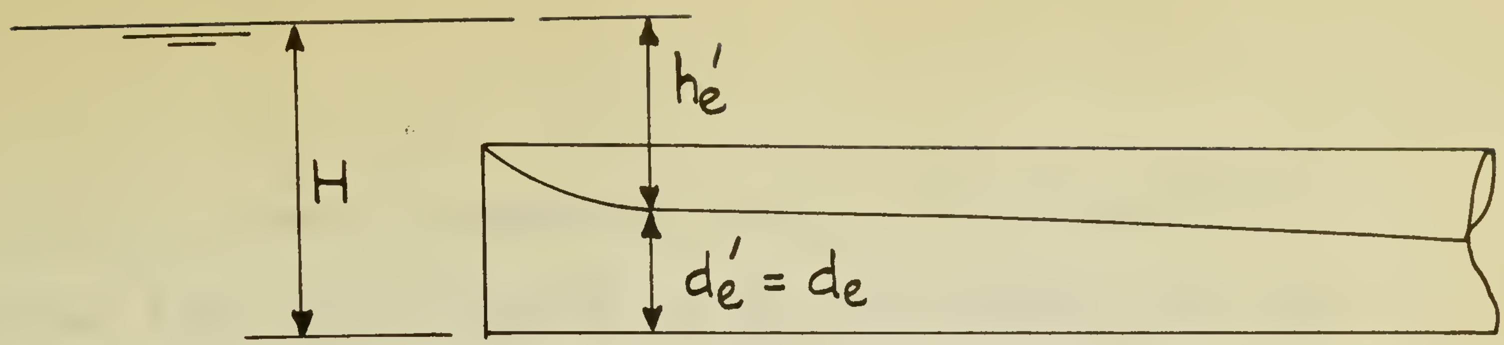
TABLE 2 - CALCULATED VALUES OF n FROM FIELD TESTS

	Test No.	n	
Uniform Sub-Critical Flow	F13	0.033	
	F18	0.033	
	F19	0.032	
	F32	0.037	
	F33	0.035	
	G1	0.035	
	G3	0.038	
	G4	0.035	
		8)0.278	Average 0.035
Uniform Critical Flow	F5	0.032	
	F6	0.030	
	F7	0.029	
	F8	0.038	
	F9	0.036	
	F10	0.036	
	F35	0.036	
	F36	0.036	
		8)0.273	Average 0.034
Non-Uniform Sub-Critical (M2 Backwater Curves)	F14	0.037	
	F15	0.034	
	F16	0.036	
	F17	0.036	
	F21	0.035	
	F22	0.036	
	F23	0.035	
	F24	0.036	
Includes full flow -	F25	0.038	average
Includes full flow -	F26	0.038	average
	F27	0.038	
	F28	0.038	
	F29	0.039	
	F30	0.038	
Includes full flow -	F31	0.038	average
	F34	0.037	
		16)0.589	Average 0.037
Super-Critical Flow (?)	F2	0.034	
	F11	0.035	

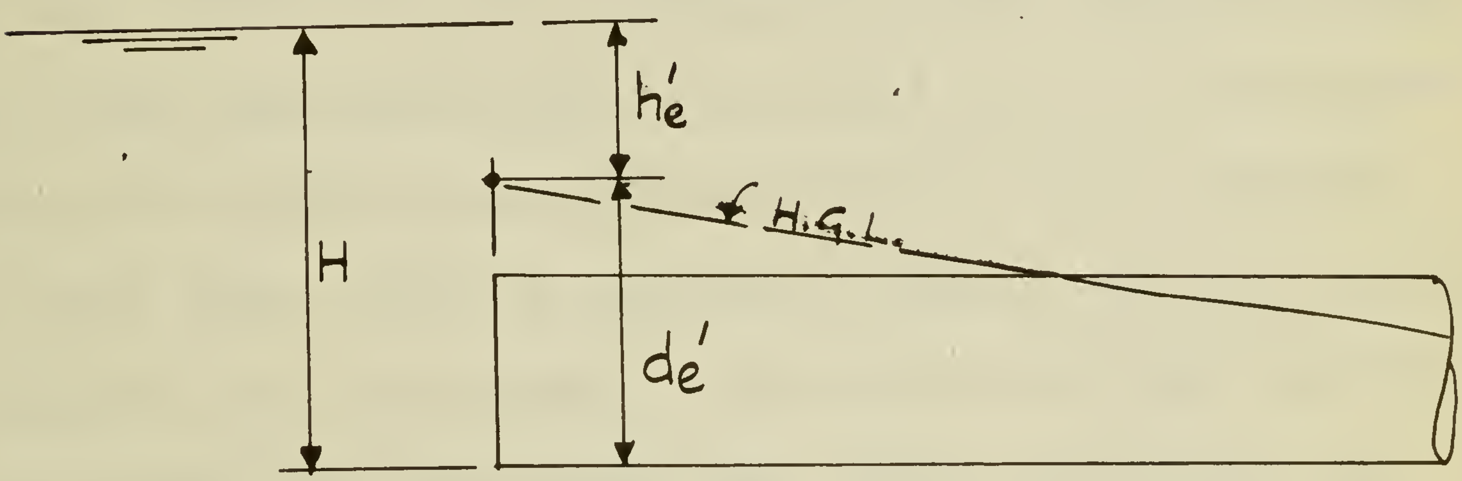
Average of all Tests: 0.036

depth of flow inside the inlet - i.e. $(H - d_e)$ - represents the true velocity head inside the pipe plus the entrance head loss. This will now be referred to as the "entrance drawdown", and given the symbol h_e'. In Chapter 2, the discussion was confined to cases where the pipe flowed part-full over its entire length, but in some of the field tests the culvert flowed full for a certain distance, and then part-full. The conception of entrance drawdown can be applied to the latter situation by using, instead of the flow depth d_e, the height of the hydraulic grade line above the invert, when produced in a straight line to the inlet. (In actuality, the H.G.L. does not remain straight near the inlet). The "equivalent inlet depth", d_e', can be defined as meaning the actual flow depth for part-full flow, and the height of the H.G.L. produced for full flow, as illustrated in Fig. 14. Since the constituents of the entrance drawdown are conventionally expressed as fractions of $V_e^2/2g$, it can also be so expressed, by introducing an "entrance drawdown coefficient", k. These relationships are made clear in Fig. 14.

On the data and analysis sheets, k was calculated for each of the field tests for which sufficient data were available. It was decided to present the values of k as calculated, rather than try to present values of the entrance loss coefficient K_e, as is usually done, since to do this it would have been necessary to assume



(a) PART-FULL FLOW AT INLET



(b) FULL FLOW AT INLET

EQUATIONS :

$$\text{ENTRANCE DRAWDOWN } h_e' = H - d_e'$$
$$= k\, V_e^2/2g$$
$$= (K_e + \alpha)\, V_e^2/2g$$

i.e., ENTRANCE DRAWDOWN COEFFICIENT $k = K_e + \alpha$

FIG. 14 - ILLUSTRATING CONCEPTION OF "ENTRANCE DRAWDOWN" (CULVERTS ON MILD SLOPES)

values of α . Table 3 shows the values of k for cases of sub-critical flow only; although on the data sheets they were worked out for some other cases, it was later decided that for critical or super-critical flow, the figures had little meaning.

Fig. 15 shows the values of k listed in Table 3, plotted against both $Q/D^{5/2}$ and d_e'/D. Although it would perhaps be more logical to plot them against H/D, this would not be of much practical value, for reasons which are explained in 3.6.4. Between the two plots shown, there is not much to choose; the first perhaps shows slightly more consistency. Curves have been drawn for the square projecting and hood inlets only, as the results for the others were too scattered to justify drawing curves. It has been assumed that the curves start at k = 1.0 when Q = 0, and converge to constant k values; this is discussed further in section 3.6.

3.5.6. Flow Profiles

The last section on each data and analysis sheet contains a note or sketch of the flow profile which occurred, or is assumed to have occurred. Some of the profiles require comment, as follows:

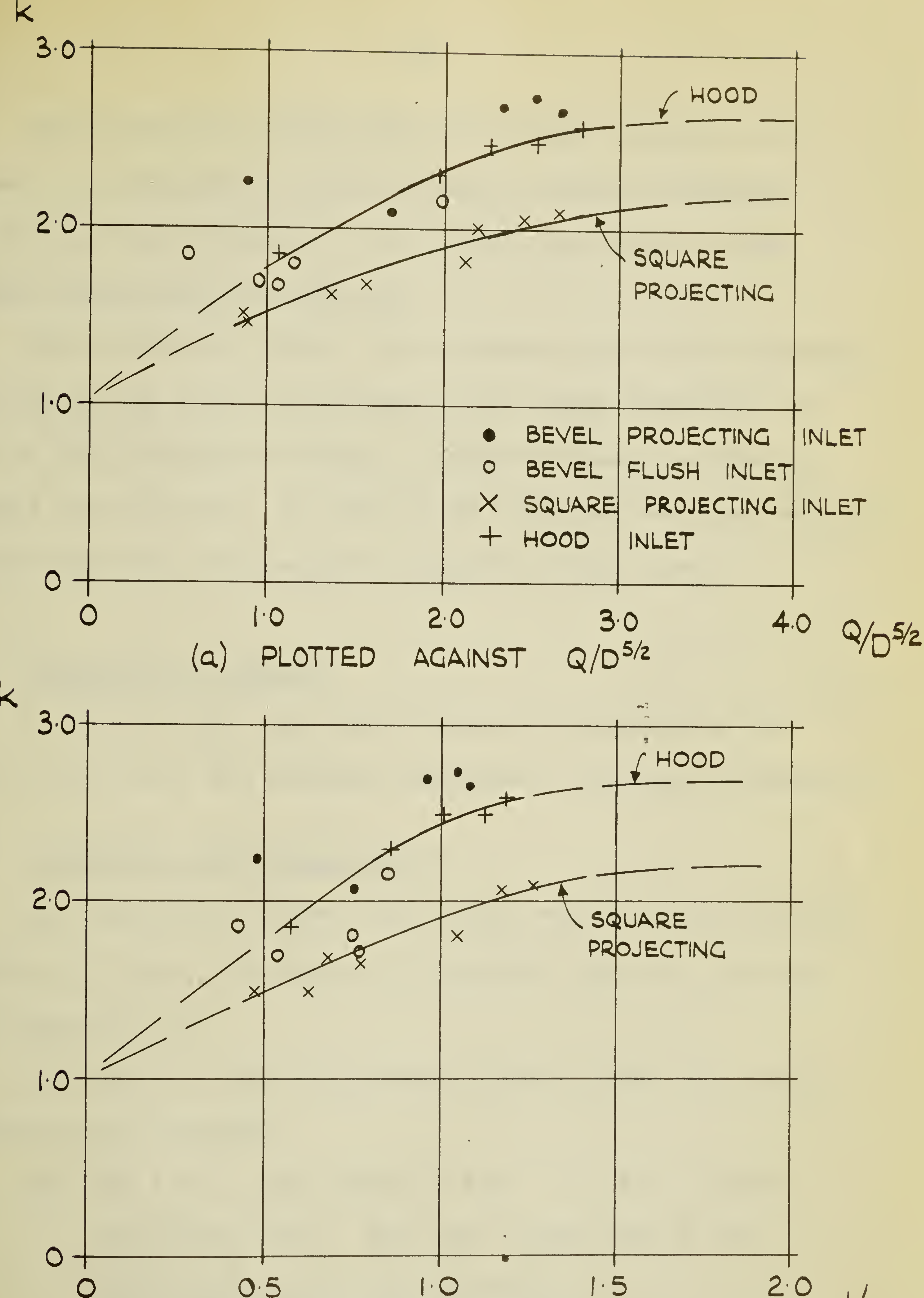
F2 and F11 - These have been sketched as S1 curves (super-critical), based on the piezometer readings, but visual observation suggested uniform depth. If uniform depth is assumed, higher n values result.

TABLE 3

CALCULATED VALUES OF ENTRANCE DRAWDOWN COEFFICIENT, k

(Tabulated for sub-critical flow only)

	Test No.	$Q/D^{5/2}$	d_e'/D	k
Bevel Projecting Inlet	F13	0.89	0.49	2.25
	F14	1.70	0.75	2.1
	F15	2.34	0.92	2.7
	F16	2.52	1.04	2.75
	F17	2.64	1.09	2.64
Square Projecting Inlet	F18	0.86	0.48	1.5
	F19	1.57	0.69	1.67
	F21	2.20	0.88	2.0
	F22	0.86	0.62	1.5
	F23	1.36	0.77	1.62
	F24	2.11	1.04	1.8
	F25	2.45	1.18	2.04
	F26	2.64	1.26	2.1
Hood Inlet	F27	1.05	0.58	1.84
	F28	1.98	0.85	2.3
	F29	2.24	1.0	2.5
	F30	2.52	1.12	2.6
	F31	2.80	1.19	2.7
Bevel Flush Inlet	F32	0.54	0.42	1.85
	F33	0.96	0.54	1.7
	F34	1.98	0.85	2.16
	G3	1.05	0.77	1.67
	G4	1.14	0.75	1.8



(a) PLOTTED AGAINST $Q/D^{5/2}$

(b) PLOTTED AGAINST d_e'/D

FIG. 15 - EXPERIMENTAL VALUES OF ENTRANCE DRAWDOWN COEFFICIENT k

F5-10, F35-36 - Uniform flow at critical depth has been assumed, but non-uniform curves may have occurred. Referring back to Fig. 3 (b), Chapter 2, it is seen that for n = 0.036, critical slope should be about 3%.

F16-17, F24-26, F30-31 - The combined mode of flow sketched, full flow at the inlet end followed by M2 curves downstream, is based on the piezometer readings, confirmed visually as far as possible (see Plate 8). In none of the tests was there any evidence of full flow with sub-atmospheric pressure at the crown.

3.6. DISCUSSION OF RESULTS

In this section, the field results are examined in the light of the theory and previous experiments discussed in Chapter 2.

3.6.1. Comparison with Design Data

The discrepancy between the results and the Armco data was expected, in view of the method of derivation used for the Armco charts (see 2.4.1.).

The deviation from the Bureau of Public Roads data can be explained on two grounds:

(1) The B.P.R. curve shown in Fig. 11 is for a square projecting inlet. The other inlets used in the tests appear to be less efficient.

(2) The B.P.R. curve is for inlet control, i.e., slopes exceeding the critical. The experimental flow profiles show that in most of the tests the culvert was on a mild slope, so that friction ought to reduce the discharge.

3.6.2. Comparison with Schiller's Model Results

In view of the last statement above, it might be thought that the field results plotted in Fig. 12 should show less relative discharge than Schiller's model results. However, it will be seen in Chapter 4 that, at H/D ratios less than 1.0, and slopes of 1% or over, the influence of friction, while theoretically significant, is of no practical importance.

3.6.3. Roughness Coefficient

The variation in the calculated values of n has already been discussed in 3.5.4. Considering the probable bias in the backwater method of calculation, it seems reasonable to apply a small negative weighting to the values so calculated, and take the overall average as 0.035. At first sight this figure appears very high, and the question arises of whether any independent check can be obtained.

As mentioned in 2.2.7., no record has been found of any previous experimental determination of n for structural plate

pipe. One very recent investigator at the U.S. Waterways Experiment Station (Vicksburg) has used fairly large scale models, and obtained a value of around 0.030, but the results have not been published yet.

There has been an extensive investigation of n for large standard C.M. pipe, (see 2.1.13.), and the value of 0.024 has been fairly well established. This value may be used to estimate the value for structural plate pipe, if it is assumed that frictional resistance in C.M. pipe follows similar laws to that in ordinary rough pipes. It may be argued, from dimensional considerations, that n should be proportional to the 6th root of the roughness height. Since the depths of the corrugations in the two types of pipe are 1/2" and 2" respectively, it follows that

$$n_{S.P.} = n_{STD.} \times (2/1/2)^{1/6}$$
$$= 0.024 \times 1.26 = 0.030$$

This answer does not seem to confirm the experimental value, but the calculation has not taken account of the different form of the corrugations in structural plate, which are not only deeper, but relatively closer together. Presumably the value calculated above should apply to corrugations 2" deep at 10-2/3" pitch, whereas the actual ones are at 6" pitch. So little is known of the mechanics of flow over corrugated surfaces, that it is impossible

pipe. The only direct investigation on the U.S. Waterways Experiment Station (Vicksburg) [illegible] models, has obtained a value of around 0.026, but the results have not been published yet.

There has been an extensive investigation of a full scale standard C.M. pipe, (see [illegible]) and the value of [illegible] has fairly well established. This value can be used to estimate the value for structural plate pipe, if it is assumed that friction resistance in f.d. pipe follows similar laws to that in ordinary rough pipes. It may be argued, from dimensional considerations, that n should be proportional to the 1/6 root of the roughness height. Since the depths of the corrugations in the two types of pipe are 1/2" and 2" respectively, it follows that

$$n_{s.p.} = n_{c.m.} \times (2/(1/2))^{1/6}$$
$$= 0.024 \times 1.26 = 0.030$$

This agrees [illegible] not [illegible] to confirm the experimental value. But the calculation has not taken account of the different type of the corrugations in structural plate, which are not only deeper but relatively closer together. Presumably the value calculated above should apply to corrugations 2" deep at 16-2/3" pitch, whereas the actual ones are at 6" pitch. [illegible] the mechanics of flow over corrugated surfaces, [illegible] it is conceivable

to say what effect this might have, but it seems reasonable to suppose that it would increase n.

In a paper on friction losses in short pipes, Keulegan(17) (1948) shows theoretically that the value of the friction factor near the inlet end of a pipe is somewhat higher than the steady value reached farther downstream, because a considerable length is required to develop full turbulence over the pipe cross-section. As a result, if the friction factor (or roughness coefficient) for a short culvert is calculated from experimental results by averaging the head loss over the length (as in this investigation), the resulting f and n values will be higher than if the experiments had been made on a long culvert. The length required to attain a steady n value is rather uncertain, but may be from 5 to 10 diameters, or up to 3/4 of the length of the experimental culvert. It is also uncertain by how much this effect would increase the apparent n value - possibly by as much as 10%.

To sum up, there is no reason for thinking that the average n value of 0.035 is improbably high. Pending further tests, it therefore seems proper to adopt it for 60" structural plate pipe.

The question also arises of what values to use for larger sizes. Although n, for a given surface texture, is supposed to be independent of conduit size, there is believed to be some evidence suggesting that it decreases slightly with increasing conduit size,

and it has been argued (Blench[18], 1957) that it should be proportional to the 12th root of D. For a 10 ft. diameter pipe, this leads to the calculation

$$n_{10'} = n_{5'} \times (5/10)^{1/12}$$
$$= 0.035 \times 0.944 = 0.033.$$

Fig. 16, based on this discussion, shows recommended values of n and f for structural plate pipe.

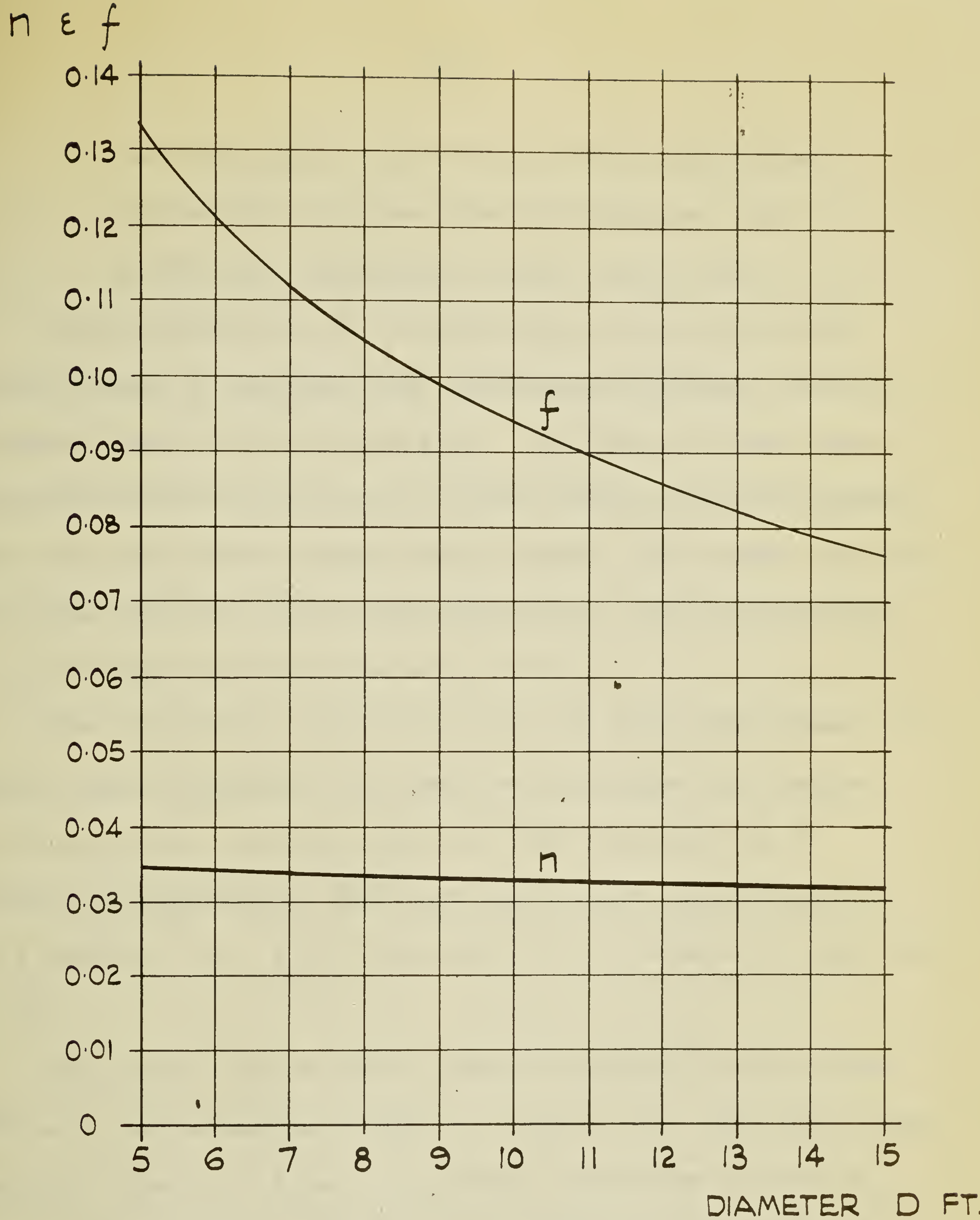
3.6.4. Entrance Drawdown Coefficients

The main purpose of plotting entrance drawdown coefficients was to provide information from which H versus Q relationships could be calculated for a culvert of any size and length on a mild slope (see 2.3.1.). The data had therefore to be plotted in such a way that, having calculated a backwater curve for a given Q and obtained d_e' (see Fig. 14), a value of k could then be taken from a graph and used to complete the calculation of H. This was the reason for selecting $Q/D^{5/2}$ and d_e'/D as parameters.

There are two probable reasons for the somewhat inconclusive results:

(1) There are not enough experimental data.

(2) The conception illustrated in Fig. 14 over-simplifies the true situation at a pipe inlet. A more rigorous analysis is possible, but would be too complicated



$$f = \frac{185\,n^2}{D^{1/3}}$$

FIG. 16 – RECOMMENDED VALUES OF n AND f FOR STRUCTURAL PLATE C.M. PIPE

(BASED ON FIELD EXPERIMENTS ON 60″ CULVERT)

for the purpose stated above. The difficulties can perhaps be appreciated by looking at some of the test photographs, e.g., Plate 4 (a).

Examining Fig. 15, it is seen that, at any rate within a certain range, k increases with increasing discharge, or with increasing depth of flow in the pipe. The points for the square projecting and hood inlets can be fitted fairly closely by smooth curves, but the others scatter fairly widely. The reason for this may be that with the square projecting inlet, all the tests were at 0 or 1% slope, and with the hood, at 1%.

The value of $k = 1.0$ for Q or $d_e' = 0$, which the curves suggest, seems reasonable. At very low discharges the contraction caused by the inlet is small, so that the head loss in turbulent re-expansion is small and the velocity distribution is nearly uniform; i.e., $K_e \doteq 0$ and $\alpha \doteq 1.0$, whence $k (= Ke + \alpha) \doteq 1.0$.

The curves seem to tend to nearly constant k values above $Q/D^{5/2} = 3$, corresponding to $H/D \doteq 2$ (Fig. 13). This also seems reasonable, since the form of the inlet contraction depends on the streamline pattern imposed by the geometry of the inlet and the headpond; at H/D ratios exceeding 2, the streamline pattern will be essentially that imposed by a reservoir of infinite depth.

[illegible]

[illegible]

[illegible]

The maximum value of k for the square projecting inlet is about 2.2. If α is taken as 1.3 (see 3.5.4.), this gives K_e = 0.9, which agrees with figures quoted by other investigators (see 2.2.3.). For the hood inlet, the maximum k value is about 2.7, which does not seem to agree too well with the K_e value of 1.0 quoted by others.

To sum up, the curve shown in Fig. 15 (b) for the square projecting inlet probably provides a reasonable basis for calculating H v. Q relationships for culverts on mild slopes. For the other inlets tested, the results are not too conclusive. The evidence suggests that, with respect to entrance losses, the square projecting inlet is more efficient than the others.

3.6.5. <u>Flow Profiles - Priming</u>

It has been seen that the flow profiles which occurred in the field tests can be analyzed by pipe and open channel theory. With certain exceptions, they can be reproduced qualitatively in smooth laboratory models, if slopes are reduced to allow for the smaller roughness coefficient of the model. The exceptions are the profiles of tests F16-17, 24-26, and 30-31, all of which show full flow at the inlet end followed by part-full flow at the outlet end. In a smooth model, such a condition is not stable; as soon as the discharge rises high enough to fill the pipe at the inlet end, the point of contact at the roof begins to move down-

The maximum value of [illegible]

[illegible]

[illegible]

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[illegible]

stream, until the pipe flows full throughout its length - i.e., the model primes. Near the outlet end of a primed culvert, the pressure at the crown is sub-atmospheric, i.e., the H.G.L. is below the crown (see 2.3.4.).

Three possible reasons for the failure of the field culvert to prime are considered, as follows:

(1) Air leakage through the joints prevented the maintenance of sub-atmospheric pressure.
Air leakage was undoubtedly considerable. During the tests enumerated above, water spouted freely from some of the joints at the inlet end of the pipe where it was running full under pressure. Probably the leakage was quite sufficient to prevent sub-atmospheric pressure developing at the outlet end.

(2) Corrugations inhibit priming.
To investigate this possibility, a transparent corrugated model culvert, scaled from the 60" prototype, was tested in the University of Alberta hydraulics laboratory. While it was possible to make the model prime, it did not do so nearly as easily as a smooth model, requiring higher heads and longer times.

(3) Priming is caused by factors which are effective only in small-scale pipes.

It is quite possible that priming may occur more easily in small pipes than in large ones; again, further experimental work, together with a theoretical explanation of the mechanism of priming, would be necessary before an answer could be given. Probably the effects of the corrugations and of the leaky joints combined to prevent priming.

An important practical question is whether the failure to prime had any serious effect on the culvert's capacity. Calculations based on the method outlined in 2.3.4. indicate that, in the tests under discussion, there would have been very little difference in headwater depth if the culvert had primed, and that in some cases H might actually have increased. Such a result seems curious, but it is possible to demonstrate it with a laboratory model, within a small range of slopes and discharges.

3.6.6. Inlet Vortices

It has sometimes been supposed that vortices at the inlet have an important effect on culvert operation. The field tests gave no support to this idea. The bevel inlet seems to induce the strongest vortices, but they were also observed with the square

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possible, would be [illegible]
would be [illegible]
[illegible]
[illegible]

An important practical question is whether the [illegible]

[illegible]

[illegible] Tidal Friction

It has [illegible]

[illegible]

inlet, although not with the hood. The quantity of air sucked in was not sufficient to affect the discharge to any noticeable extent. Similar effects have been observed on models.

3.7. CONCLUSIONS FROM FIELD EXPERIMENTS

With respect to the objects of the investigation, as stated in Chapter 1, the principal conclusions drawn from the field experiments were as follows:

(1) The discharge figures usually taken from the Armco handbook for design purposes are too high.

(2) The U.S. Bureau of Public Roads nomographs are adequate for the design of short culverts at low heads.

(3) The roughness coefficient n for structural plate C.M. pipe is about 0.035.

(4) Bevel inlets have no hydraulic advantages, and appear to be less efficient than square projecting inlets.

(5) Hood inlets offer no advantages for large C.M. highway culverts.

(6) A new form of design chart should be developed for C.M. culverts on mild slopes, which would enable discharges to be predicted for a wide range of lengths and heads.

CHAPTER 4 - DESIGN CHARTS FOR LARGE CORRUGATED-METAL CULVERTS

A method of calculating head-discharge relationships for culverts on mild slopes, taking account of slope, length and roughness, has already been indicated in various parts of Chapters 2 and 3. In this chapter it is presented systematically, and used for the preparation of a new form of design chart for large C.M. culverts.

4.1. JUSTIFICATION OF NEW CHARTS

The analysis of the field experiments described in Chapter 3 showed that the Armco charts were incorrect, and that the B.P.R. nomographs, while not seriously inaccurate for the range of conditions covered in the experiments, might be quite wrong for lengths and heads outside the experimental range. The n value found for structural plate pipe showed that slopes of between 0 and 3%, which are very common in highway practice, were definitely mild, so that acceptable design charts ought to take account of length, slope and roughness.

4.2. CALCULATION OF HEAD-DISCHARGE RELATIONSHIPS

The method is essentially that outlined in 2.3.1., and used for the analysis of many of the field tests. The steps are as follows:

1. Select diameter D, slope s_o, length L, roughness n, discharge Q.
2. Calculate critical depth d_c, using Table 1.
3. Assume that depth of flow at outlet = critical depth, and do step backwater calculation to get depth of flow at length L from the outlet, i.e., d_e. If backwater curve hits the culvert roof at length $l < L$, calculate head loss for full flow over length (L-l), to get height of H.G.L. at inlet, i.e., d'_e.
4. Take k from Fig. 15 and multiply by $V_e{}^2/2g$ to get entrance drawdown h'_e.
5. Add h'_e to d'_e to get H, which gives one point on a rating curve.
6. Repeat for a series of Q values and draw one rating curve for one length, slope, and roughness.

The procedure is rather laborious, but in practice the same backwater calculation can be used for a series of different lengths. A sample calculation, for a 5' diameter culvert on a

slope of 1%, at Q = 120 c.f.s., is shown in Table 4. It should be noted that both the critical depth and the velocity head are corrected by inclusion of the K.E. correction factor α , taken as 1.3.

4.3. DESIGN CHARTS

It was decided to present the design data meanwhile in the form of rating curves. This entails, for clear reading, a separate chart for each diameter and slope. Nomographic charts would be much more compact, but, besides being more troublesome to prepare, they do not give the same graphic illustration of the relationships between different factors.

Figs. 17 and 18 show a set of charts for 60" culverts on slopes of 0, 1%, 2% and 3%. They cover lengths of 70' to 300', and headwater depths up to 15'. The applicable field results have been plotted on Fig. 17; the points fall almost exactly on the curve for the 70' length. The curves have been prepared using the experimental values of k for the square projecting inlet, but may be used also for bevel flush inlets, since the difference in discharge should not amount to more than a few percent.

Comparison of the curves brings out some interesting points. On zero slope, the effect of length is noticeable at all stages, but on 1% slope, it is only noticeable at depths exceeding 5', and

TABLE 4

Given: $D = 5'$, $S_o = 1\% = 0.010$, $Q = 120$ c.f.s., $n = 0.035$.

$Q/D^{5/2} = 2.14$. From Table 1, $d_c/D = 0.625$.
Corrected $d_c = 0.625 \times 5 \times \sqrt[3]{1.3} = 3.4'$.

By Manning's formula, $S_f = \left(\frac{Qn}{1.49\ A\ R^{2/3}}\right)^2 = \frac{7.95}{(A\ R^{2/3})^2}$

Corrected velocity head $= \alpha \frac{V^2}{2g} = \frac{1.3\ Q^2}{2g\ A^2} = \frac{291}{A^2}$

$E_f = d + \alpha \frac{V^2}{2g}$

$\Delta L = \frac{\Delta E_f}{S_f - S_o}$

Backwater Calculation:

d	A	R	$R^{2/3}$	$\alpha V^2/2g$	E_f	ΔE_f	S_f	$S_f - S_o$	ΔL	L
3.4	14.2			1.44	4.84					
3.8	16.0	1.51	1.32	1.14	4.94	0.10	.0178	.0078	13	13
4.1	17.2	1.52	1.32	0.98	5.08	.14	.0154	.0054	26	39
4.4	18.3	1.50	1.31	0.87	5.27	.19	.0138	.0038	50	89
4.6	18.9	1.46	1.29	0.81	5.41	.14	.0134	.0034	41	130
4.8	19.4	1.41	1.26	0.77	5.57	.16	.0133	.0033	48	178
5.0	19.6	1.25	1.16	0.75	5.75	.18	.0153	.0053	34	212

Headwater Depth Calculation: Lengths 70', 150', 300'.

l_f = length running full. $d_e' = d_e + l_f\,(S_f - S_o)_{d=5'}$

$H = d_e' + h_e'$

(Assume square projecting inlet) $h_e' = k\ V_e^2/2g$

L	d_e	l_f	$l_f(S_f-S_o)$	d_e'	k	$V_e^2/2g$	h_e'	H
70'	4.3	-	-	4.3	1.8	0.69	1.24	5.5
150'	4.7	-	-	4.7	1.9	0.61	1.16	5.9
300'	5.0	88	0.47	5.47	2.0	0.58	1.16	6.6

TABLE 4 - SAMPLE CALCULATION SHOWING METHOD OF DERIVING H-Q RELATIONSHIPS

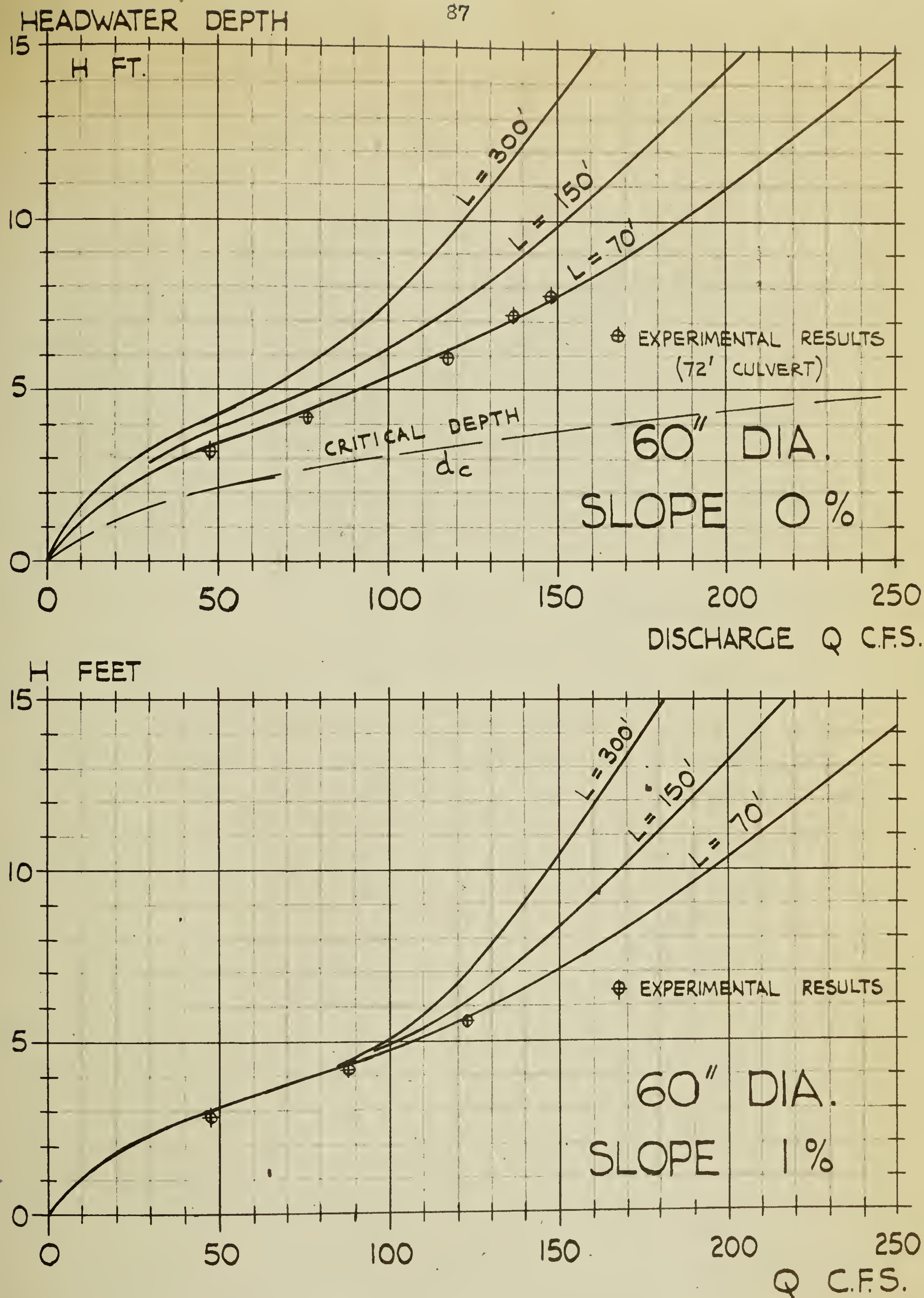
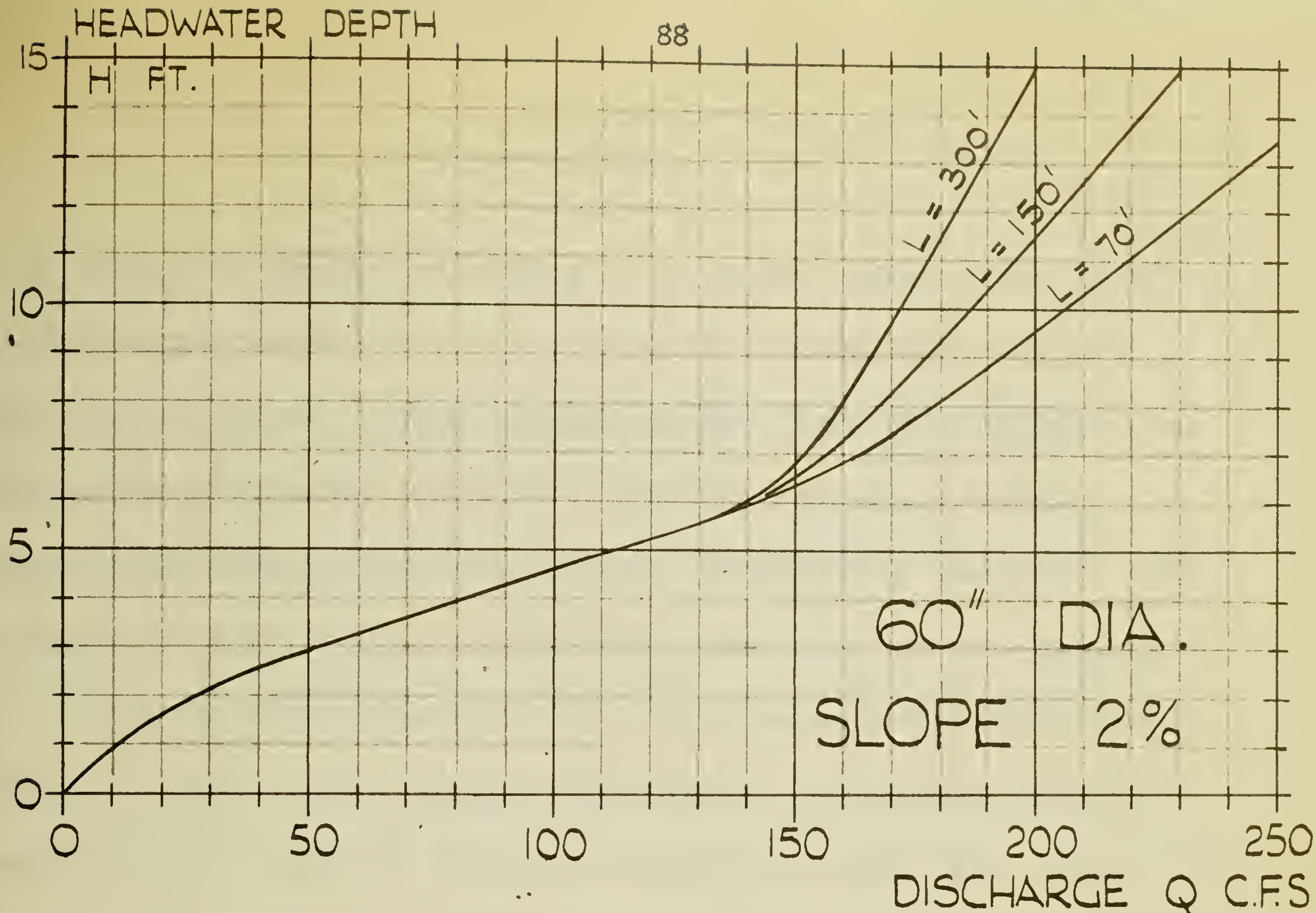


FIG. 17 - CULVERT DESIGN CHARTS.
SQUARE PROJECTING INLET, FREE OUTLET.
$n = 0{\cdot}035$.



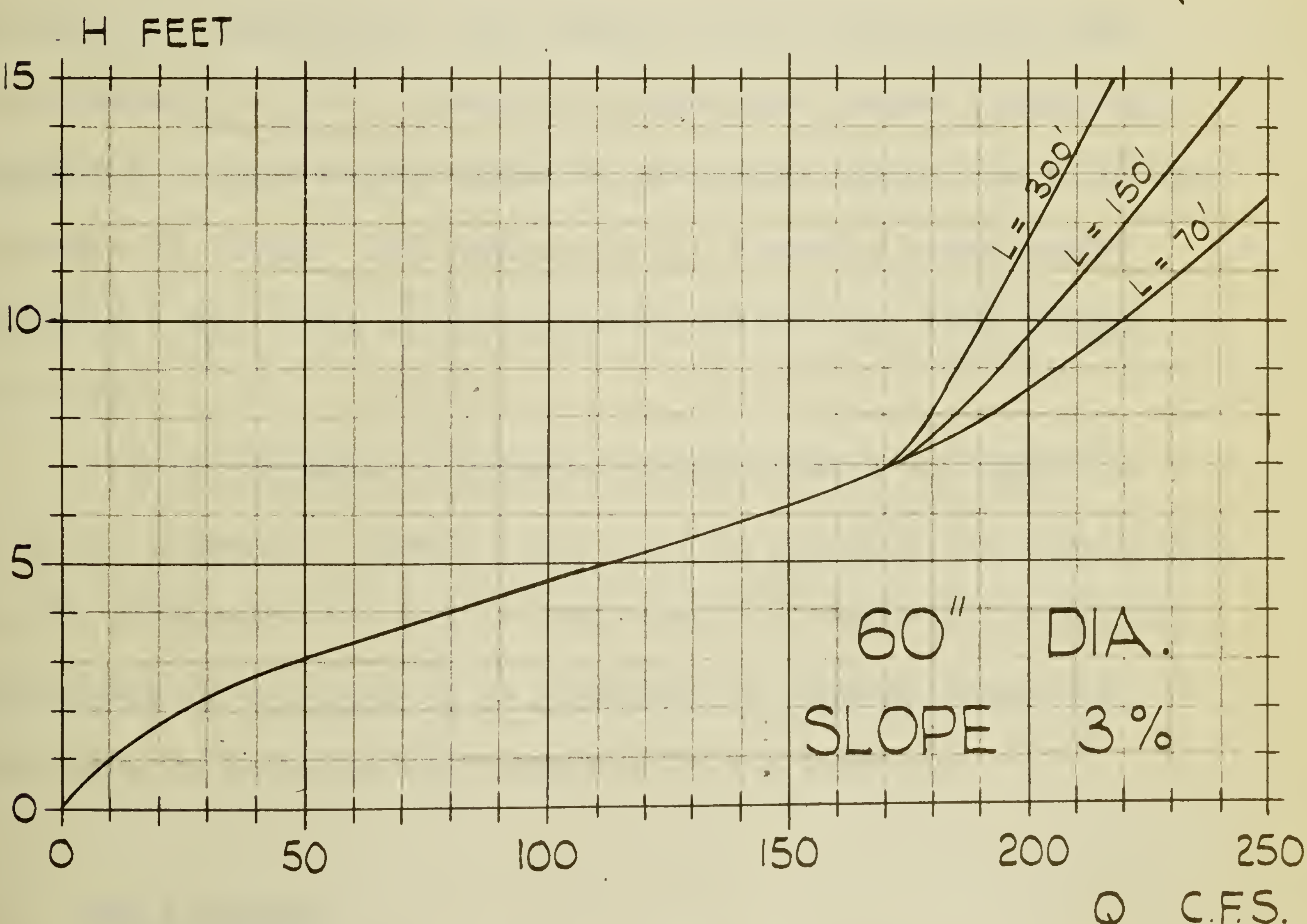


FIG. 18 - CULVERT DESIGN CHARTS.
SQUARE PROJECTING INLET, FREE OUTLET.
$n = 0.035$.

on 3% slope, at depths exceeding 7'. At all slopes over 1%, the discharge for H = 5', i.e., inlet just submerged, may be taken as 110 c.f.s. At the higher slopes, the 300' length shows a steeply rising curve after the discharge passes a certain figure; this sharp change in the H/Q relationship marks the point at which the pipe begins to flow full over most of its length.

On the first chart of Fig. 17, a curve has been plotted showing critical depth against discharge, the purpose being to show the upper limit of tailwater depths for which the charts are strictly applicable. For example, if Q = 100 c.f.s., the curve shows $d_c = 3.8'$. Provide the tailwater depth T does not exceed 3.8', the H values given by the chart apply, i.e. 5.2' for a culvert 70' long. If T exceeds 3.8', H may be corrected by adding $(T - d_c)$; this is not strictly correct but errs on the safe side.

It is intended to prepare similar charts for pipes up to 10 ft. diameter. Charts could also be prepared for pipe-arches, but the difficulty in this is that areas and hydraulic radii for pipe-arches do not seem to be available in tabular form, and would have to be worked out graphically for each size.

4.4. USE OF CHARTS

In highway culvert design, the usual procedure is to select a pipe which will carry a 10-year to 20-year estimated

[illegible] As any [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] at the [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] at which the [illegible] [illegible] [illegible] [illegible] [illegible] [illegible].

On the [illegible] [illegible] of [illegible].

It is [illegible] to [illegible].

5.4. [illegible]

[illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible]

flood with the headwater just submerging the inlet, i.e., H = D. The selected size should then be checked to see whether the headwater caused by an estimated 100-year flood would top the highway embankment. Since the charts go up to H/D = 3, they should be adequate for most highway installations.

The charts should also be useful for hydrologic purposes, in estimating actual stream flows from observed flood levels at culverts. It should be remembered, however, that they apply, strictly speaking, only to culverts with free outlets, i.e., with the tailwater depth less than the critical depth in the pipe.

CHAPTER 5 - PROBLEMS IN FLUID MECHANICS

The investigations described raise many interesting problems in fluid mechanics, two of which are discussed briefly in this chapter.

5.1. PRIMING

The priming of a smooth transparent model culvert 3-1/2" diameter x 80" long has been observed closely in the University of Alberta hydraulics laboratory. It was concluded that priming from the inlet end, with a free outlet, is initiated in all cases by making the flow touch the roof of the culvert near the inlet. The initial sealing at the roof may be accomplished in different ways, as follows:

(a) If the culvert is on a mild slope, and is sufficiently long, the backwater curve, controlled by critical depth at the outlet, may rise high enough to touch the roof at the inlet end.

(b) If a hood inlet is fitted, the streamlines of the entering flow are directed upwards to strike the roof.

(c) If a bellmouth inlet is fitted, the streamlines are directed so that they touch the pipe circumference all round.

(d) If the headwater is sufficiently high, the highly turbulent flow pattern inside the inlet may throw up enough water to seal the pipe, regardless of the slope or type of inlet.

These 4 ways of initiating priming are illustrated in Fig. 19. The diagrams are of course merely rough representations of 3-dimensional phenomena.

Once priming of the smooth model is initiated, the pipe generally fills rapidly, the priming "wave" traversing the length of the pipe in a few seconds. The wave front maintains a constant angle with the pipe roof, as shown on Fig. 20 (a). It has been noticed, however, that the process is sometimes erratic, and that, particularly if the roof is dry, the wave may progress in spurts. A surface irregularity in the roof may cause the wave to stop there until the discharge is increased. Metzler and Rouse[11] (1959) note that applying grease or a wetting agent to a model can retard or advance priming.

After priming is complete, piezometer taps in the culvert roof indicate sub-atmospheric pressures near the outlet end. (On steep slopes, the entire length may have sub-atmospheric pressures). The question arises of what causes the priming wave to advance into a region of sub-atmospheric pressure, or more simply, why the water rises above its natural level. The answer seems to be that the air above the water must be extracted faster than it can be replaced from downstream. Experiments with smoke indicated a

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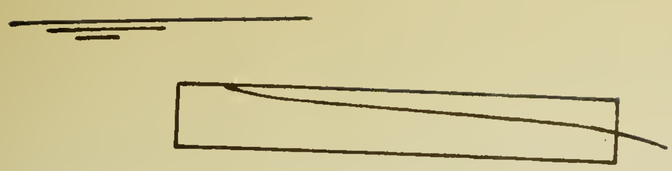
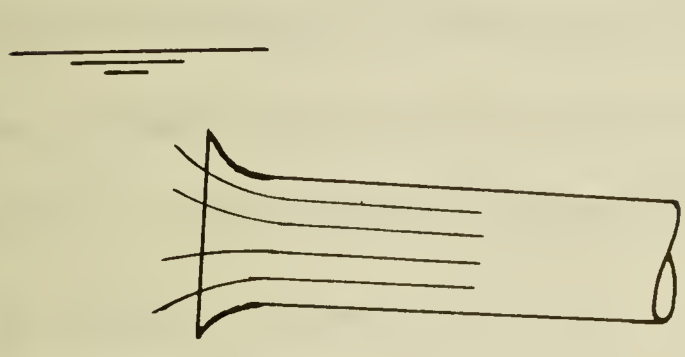
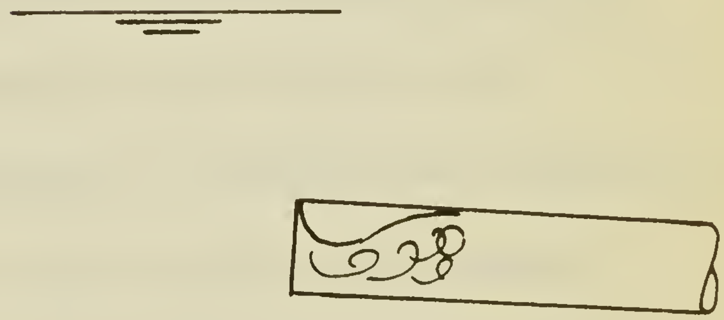


(a) BACKWATER CURVE TOUCHES ROOF (MILD SLOPE)

(b) HOOD INLET DIRECTS STREAMLINES UPWARDS

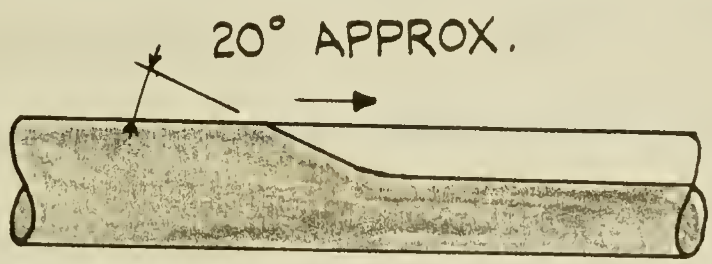


(c) BELLMOUTH DIRECTS STREAMLINES PARALLEL

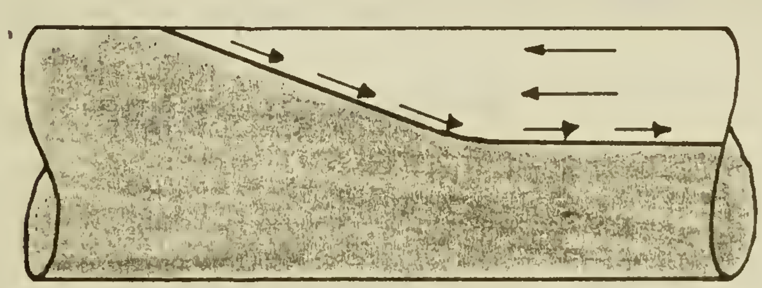


(d) HIGH HEAD THROWS FLOW TO ROOF

FIG. 19 - 4 WAYS OF INITIATING PRIMING



(a) PRIMING WAVE MOVING DOWNSTREAM



(b) AIR CIRCULATION DURING PRIMING

FIG. 20 - PRIMING OBSERVATIONS ON 3½" DIA. SMOOTH MODEL

circulation as shown in Fig. 20 (b), a very thin layer of air moving rapidly in contact with the water, and the remainder of the air space moving slowly in the opposite direction. This suggests that suction is induced in the narrow space where the top of the priming wave meets the roof, causing the wave to keep advancing. The effects caused by greasing or wetting the surface, and by surface irregularities, suggest that surface tension is also influential.

Observations were also made in the University laboratory on a model of similar dimensions to the smooth one, corrugated internally to represent 60" structural plate pipe to scale. This model did not prime so readily as the smooth one; after priming was initiated, it generally took several minutes to complete, and at low headwater depths it did not complete at all. It was evident that the corrugations trapped air at the roof, and that at low velocities all the air could not be extracted. This lends support to the suggestion advanced above concerning the mechanism of priming, since a corrugated roof does not offer the same narrow space favorable to the development of suction.

5.2. FRICTION IN CORRUGATED PIPES

Limitations of space and time prevent any general discussion of pipe friction, and familiarity with the general findings on the subject must be assumed.

Considering the long time corrugated-metal pipes have been in use, and the large number of experiments done on pipe friction, it is curious that so little work has been done on the mechanics of resistance in corrugated conduits. The reasons are probably that the use of corrugated-metal pipes has been restricted mainly to culverts, which are seldom important hydraulic structures, and to North America, whereas much of the fundamental work has been done in Europe. Reference has been made by several writers to experiments by Hopf and Fromm in 1923, on pipes with corrugated surfaces, but it appears that their corrugations were relatively so flat that no separation of flow occurred, and that their results are not applicable to commercial pipe. The most important published work on commercial pipe seems to be that of Webster and Metcalf[12] (1959), which was more of a practical than a fundamental investigation.

Three categories of turbulent pipe flow have been recognized - smooth pipe, transition, and rough pipe (or fully-developed turbulent). In the first, the friction factor f depends only on Reynold's number R_n, in the second it depends both on R_n and on relative roughness k/D (where k is a representative roughness height or depth), and in the third it depends only on k/D. The physical meaning is supposed to be that in the first case the roughness elements are very small compared to the thickness of the laminar

layer on the wall, in the second they project into it, and in the third they project through it into the turbulent flow. Since the thickness of the laminar layer decreases as R_n increases, a pipe may be smooth at low R_n and rough at high R_n. In smooth pipe flow, f decreases as R_n increases; in transition flow it may increase or decrease. Manning's formula is a rough pipe formula, if n is taken as a constant for a given pipe.

The classic experiments on artificially roughened pipes were those of Nikuradse[19] (1933), who used pipes coated with closely-packed uniform sand grains set in varnish. In a largely speculative paper on the mechanics of pipe friction, Morris[23] (1955) describes flow over both corrugations and sand-grains as typical "wake-interference flow". It is therefore interesting to compare the available data for commercial C.M. pipe with Nikuradse's. Fig. 21 shows, together with Nikuradse's curves for the rougher pipes, curves published by Straub and Morris[4] (1950) and Webster and Metcalf[12] (1959) for standard C.M. pipes from 18" to 84" diameter. Also shown is a short broken line representing the average result from the field experiments, and a single point representing a preliminary test on the model corrugated pipe previously mentioned; testing of this pipe at higher Reynold's numbers has not been possible in time for this report.

The shape of the curves for standard C.M. pipes in Fig. 21 suggests transitional flow, although intuition suggests that the

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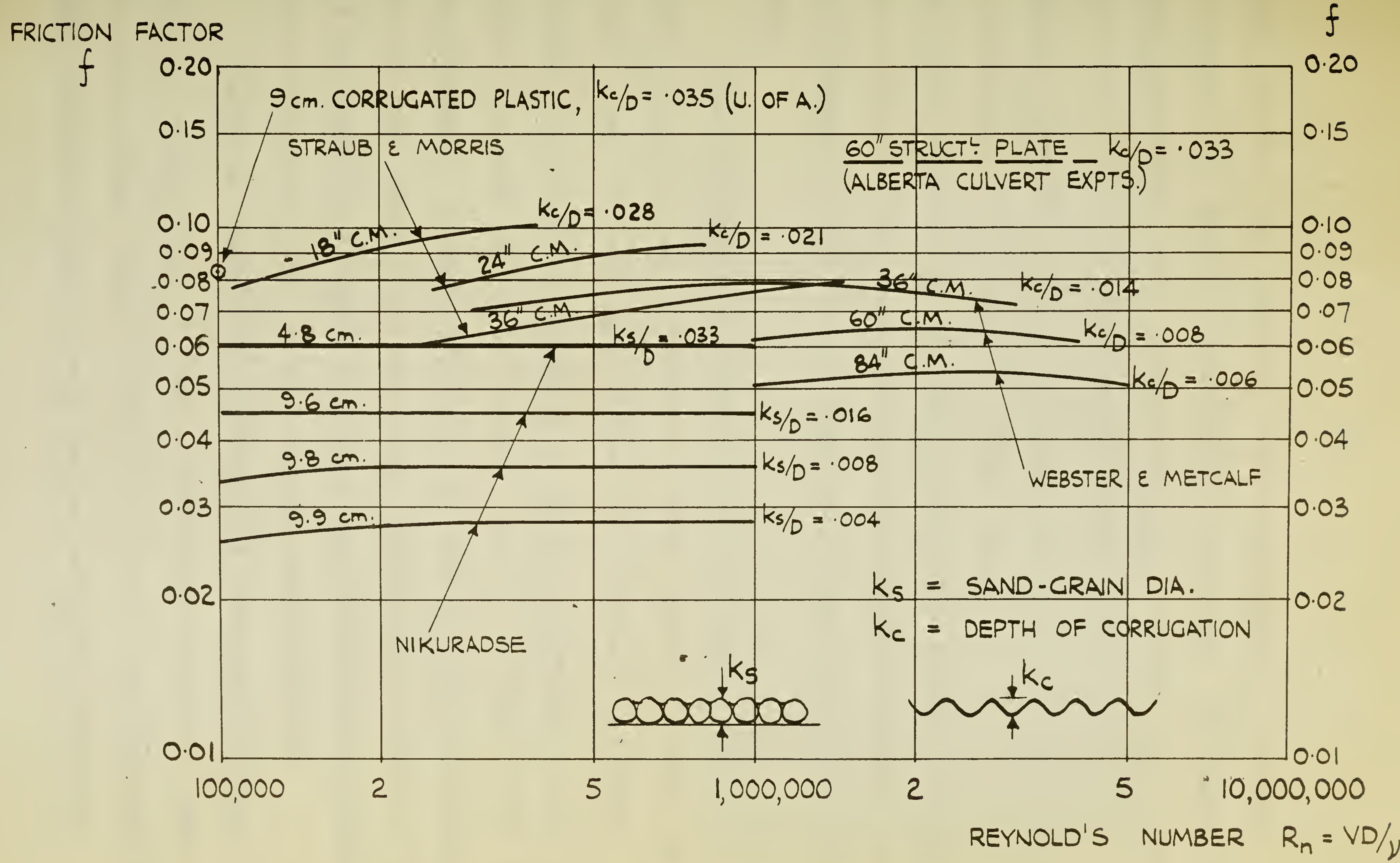

FIG. 21 – FRICTION FACTORS FOR C.M. PIPE COMPARED WITH NIKURADSE'S RESULTS FOR SAND-GRAIN ROUGHNESS

highly disturbed flow observed in corrugated pipes at moderate velocities is in effect fully developed turbulence. The equivalent sand-grain diameter for a standard corrugated surface seems to be about 2 to 4 times the depth of corrugation, which appears reasonable when the effective roughness depth of varnished spheres is considered. Schlichting[20] (1955) quotes the case of a water main 50 cm. diameter, which became covered with a deposit in the form of fine rib-like corrugations; the calculated equivalent sand-grain diameter is said to have been over 25 times the actual rib height.

A unique feature of corrugated pipe is that it has a periodic expansion and contraction of area, which is not the case in other types of rough pipe, whether the roughness is regular or random. This might be expected to produce pulsations in the flow. In the field tests on the 60" culvert, steady vibration of the pipe was noticeable. It seems that the area used in flow calculations should be the lesser area, and not the average area, which is often used for conduits of random roughness, such as unlined rock tunnels. All the data in Fig. 21 are based on the lesser diameter.

Purely as a matter of interest, the photograph on Plate 11, taken in the University of Alberta hydraulics laboratory, shows the optical fringe pattern obtained in a fluid polariscope from a doubly refractive liquid flowing between corrugated boundaries.

PLATE 11 - OPTICAL FRINGE PATTERN PRODUCED IN FLUID POLARISCOPE BY LAMINAR FLOW BETWEEN CORRUGATED BOUNDARIES (UPPER PART OF PHOTOGRAPH). FLOW LEFT TO RIGHT.

The fringe pattern, which in the form illustrated denotes laminar flow, is determined by shear stresses in the fluid, and may be used to obtain velocity distributions [21]. It is not certain whether it has any practical application to the problem in hand.

CHAPTER 6 - CONCLUSIONS AND RECOMMENDATIONS

6.1. GENERAL CONCLUSIONS

Conclusions drawn from the field experiments were listed in 3.7. The following are more general conclusions drawn from the entire investigation:

(1) The suspicion which prompted the investigation, that existing design information for corrugated-metal culverts was inadequate and inaccurate, was substantially justified. The error to be expected in using the information was probably smaller than the error involved in estimating flood discharge, but this is no reason for using unreliable data on culvert capacity if more correct information can be obtained.

(2) The main fault of previous design charts was that they failed to take account of all the variables. The new charts made possible by the investigation are designed to remedy this.

(3) The hope of increasing the capacity of C.M. culverts substantially by design improvements is not likely to be realized. In smooth culverts, improvement of the inlet can sometimes result in a large increase in capacity. In corrugated culverts it usually has quite small effects, mainly because of the high frictional

resistance of the barrel. A bellmouth inlet (which has not yet been tested in the field) offers the best possibility, but the prediction is ventured that test results will show its advantage to be small and probably insufficient to justify its expense.

(4) Model experiments, while very useful for narrowing down the range of full-scale experiments, are no substitute for the latter, and are unreliable quantitatively for culverts. Probably much of the time and money devoted to culvert model experiments in recent years could have been spent to better advantage in the field.

(5) To compare C.M. with concrete culverts was not one of the direct objects of the investigation, but a short statement on the subject is desirable, in view of the misleading comparisons made by manufacturers. It is impossible to make any general statement about their relative hydraulic capacities, as the relationship depends on so many variables. Two culverts of a certain size, length, slope, headwater depth, and tailwater depth, but of different materials, might have identical capacities, and yet in the case of two others of a different size, length, etc., the capacity of the concrete one might be twice that of the C.M. one. The only solution is to have reliable data available for a wide range of conditions in both types of pipe, and to make a separate comparison for each design case.

6.2. RECOMMENDATIONS FOR CULVERT DESIGN

This investigation was not concerned with many of the practical problems of culvert design; for a detailed discussion of many of those, the highway engineer should consult Reference 22. The following points are, however, put forward for consideration:

(1) Unless appearance is important, C.M. culverts should have square inlets projecting to the toe of the embankment.

(2) For minimum headwater elevation at a given discharge, it is theoretically advantageous to sink the culvert invert below the stream bed. If the culvert invert is located at the stream bed elevation, the tailwater depth on most streams (at design discharge) will be considerably less than the depth of flow at the pipe outlet, i.e., the critical depth. Therefore, if the pipe is dropped until $T = d_c$, the headwater elevation will be reduced by the amount of the drop. Further dropping of the pipe is not of great advantage, but minimum headwater elevation will be achieved with the outlet completely submerged and the pipe running full.

On streams carrying bed-load, dropping the culvert invert below the bed may be unwise, since low flows will deposit sediment, which may not be scoured out at high flows. Tilton and Rowe[22] (1943) discuss this point and conclude that "with few exceptions, the bottom of the channel is the best location hydraulically, the

invert coinciding with the channel bed". The problem of ice blockage is also likely to be more acute with a sunk invert.

On irrigation canals, or sluggish streams with negligible bed-load, the above objection does not apply, and the culvert should be designed for complete submergence at design discharge. The method of calculation of 2.3.3., or charts like Fig. 9, should be used. Fig. 16 supplements Fig. 4, referred to in 2.3.3.

The culvert slope should be in most cases the same as the natural stream slope. Canal culverts should be horizontal. Camber is not recommended unless settlement is definitely expected.

(3) Concrete culverts with free outlets should be designed from the Bureau of Public Roads charts for entrance control (see 2.4.3.), unless they are very long on flat slopes.

(4) Where culverts do run submerged, as in irrigation canals, provision of a flared outlet might be economical in certain circumstances. In particular, where an existing culvert has to be lengthened to accommodate a highway widening, it may be possible to offset the increase in friction head loss by providing a bellmouth inlet and a flared outlet, thus avoiding replacement of the culvert.

6.3. RECOMMENDATIONS FOR FURTHER EXPERIMENTS

The recommendations in this section are for further work with practical objectives.

6.3.1. Strathmore Site

(1) A series of tests should be made on the existing 60" C.M. culvert, with a bellmouth inlet fitted. One slope, say 1%, with a full range of heads, would probably give enough data.

(2) The culvert could be doubled in length to 150', to enable "n" to be determined more accurately, and to give a check on the extrapolation involved in the new design charts. To avoid extra structural work, the culvert might be fixed at 1% slope.

(3) The weir might be raised to permit tests with a submerged outlet. It is doubtful if this would be worth while, since it is felt that the results can be closely predicted.

(4) A smooth liner could be fitted, to simulate concrete pipe, and the tests repeated. The same comment as in (3) applies.

(5) Tests might be made on a corrugated-metal pipe-arch.

In any further tests, the instrumentation should be improved. Piezometer taps should be provided both at the tops and the bottoms of the corrugations, and the readings compared. The manometer tubes should be enlarged to dampen the fluctuations. Pitot tube traverses might be made to obtain velocity distributions (see Ref. 12). Holes should be provided at intervals in the top of the culvert for direct depth readings.

6.3.2. Laboratory Experiments

The least satisfactory data from the 1960 experiments are those on entrance drawdown coefficients. Laboratory experiments should concentrate first on obtaining more data on this point, and on trying to find a rational basis for predicting the coefficient. The 9 cm. model is probably too small for this, but a 12" or 15" standard C.M. pipe could be used in the University of Alberta hydraulics laboratory. A complete set of discharge tests at different lengths and slopes could be made on the same pipe, up to high headwater depths, and compared with results predicted by the design chart method outlined in Chapter 4, to check the validity of extrapolating the 1960 results to other sizes and higher heads. Experiments could also be made on a small standard pipe-arch.

6.4. SUGGESTIONS FOR BASIC RESEARCH

A few topics for basic research, arising out of this investigation, are suggested as follows:

(1) Priming.

(2) The mechanics of corrugated pipe friction. A series of pipes with different sizes and shapes of corrugations could be machined fairly easily from smooth acrylic pipe.

(3) Entrance losses in pipes flowing part-full and at low heads.

(4) The hydrodynamics of culvert inlet shapes.

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APPENDIX

Each of the 41 data and analysis sheets following refers to one field test. The readings and calculations on the sheets are explained in sections 3.3. and 3.5.

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F1 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H ≈ 2.6' H/D = 0.52
Tailwater T : 2.3'
Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 0.40
Add for approach velocity + —
Corrected head Q = 25 c.f.s.

$Q/D^{5/2}$ = 0.45 Critical depth d_c = 1.35'

PIEZOMETERS								
Nº	1	2	3	4	5	6	7	8
Reading	Flow	not	deep	enough	for	reliable	readings	
Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e	$h_e' = H - d_e$	$V_e^2/2g$	$h_e' \div V_e^2/2g$

FLOW PROFILE :

INSUFFICIENT DATA RECORDED FOR CALCULATION OF n

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F2 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 3·3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 3·5' H/D = 0·70
Tailwater T : 2·4'
Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 0·62
Add for approach velocity + –
Corrected head Q = 49 c.f.s.

$Q/D^{5/2}$ = 0·875 Critical depth d_c = 1·9'

PIEZOMETERS Nº	1	2	3	4	5	6	7	8
Reading	2·65	1·95	1·55	1·2	0·75	Not readable		
Calc'd depth	2·25	1·9	1·85	1·85	1·7			

SPECIAL OBSERVATIONS : Night observation. Piezometer readings possibly incorrect.

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

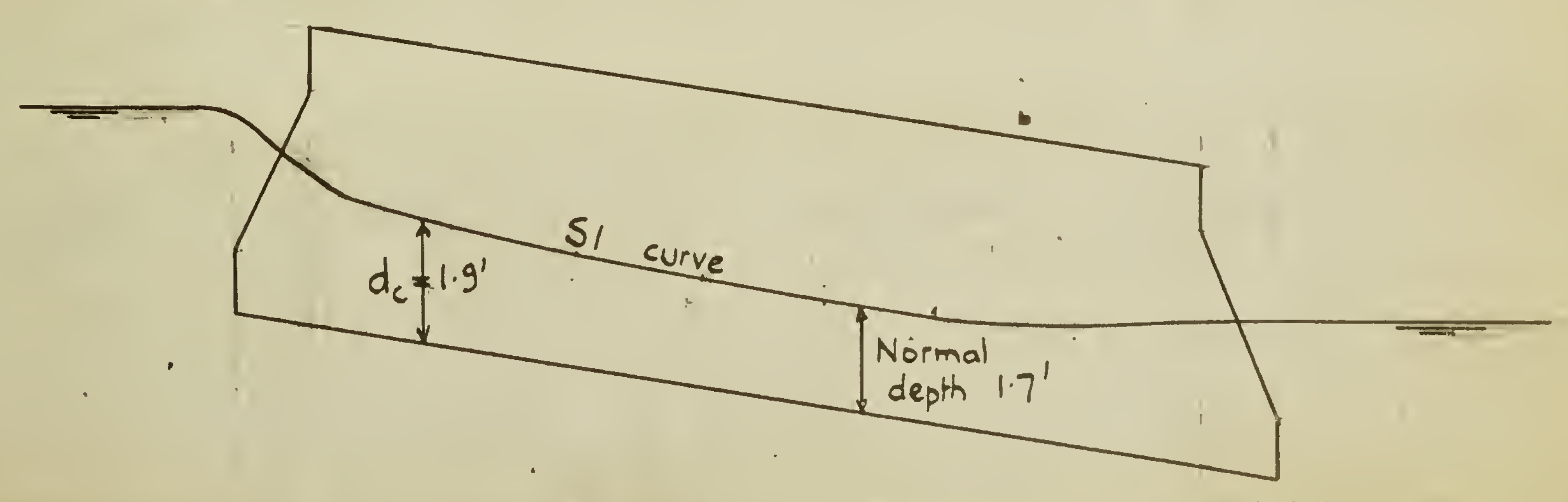
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
1·7	6·2	0·98	7·9	0·181	0·034

ENTRANCE DRAWDOWN CALCULATION :

d_e	$h_e' = H - d_e$	$V_e^2/2g$	$h_e' \div V_e^2/2g$
Data not considered sufficiently reliable			

FLOW PROFILE : (ASSUMED)



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F3 LENGTH : 72' net INLET : Bevel Projecting

DIA. : 60" SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 3.8' H/D = 0.76

Tailwater T : 2.5'

Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 0.67'

Add for approach velocity + .01

Corrected head 0.68 Q = 56 c.f.s.

$Q/D^{5/2}$ = 1.0 Critical depth d_c = 2.0'

PIEZOMETERS :

Nº	1	2	3	4	5	6	7	8
Reading	2.85'	2.05	1.7	1.35	0.9	0.5	Not	read
Calc'd depth	2.45'	2.0	2.0	2.0	1.85	1.8		

SPECIAL OBSERVATIONS : Readings on piezometers possibly incorrect

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$

$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e	$h_e' = H - d_e$	$V_e^2/2g$	$h_e' \div V_e^2/2g$

FLOW PROFILE :

NOTE : THIS TEST REPEATED APPROXIMATELY AS F 11.

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F4 LENGTH : 72' net INLET : Bevel projecting

DIA. : 60" SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 4.05' H/D = 0.81

Tailwater T : 2.5'

Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 0.76'

Add for approach velocity + 0.01

Corrected head 0.77 Q = 67 c.f.s.

$Q/D^{5/2}$ = 1.20 Critical depth d_c = 2.2'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading		Not	read					
Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = $S_f = \quad /(AR^{2/3})^2$

$V^2/2g = \quad /A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e	$h_e' = H - d_e$	$V_e^2/2g$	$h_e' \div V_e^2/2g$

FLOW PROFILE : No data.

CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F5 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 2% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 3·7' H/D = 0·74
Tailwater T : 1·3'
Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 0·70
Add for approach velocity + 0.01
Corrected head 0·71 Q = 60 c.f.s.

$Q/D^{5/2}$ = 1·07 Critical depth d_c = 2·1'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	2·95	2·15	1·9	1·6	1·4	1·3	0·85	0·65
Calc'd depth	2·7	2·1	2·05	1·95	1·95	2·05	1·8	1·8

SPECIAL OBSERVATIONS : Piezometer readings believed to be inaccurate.

BACKWATER CALCULATION : Assumed n =
S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2·1	8·33	1·10	7·2	0·141	0·032

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2·1	1·6	0·81	= 2·0

FLOW PROFILE :

Uniform critical flow assumed.

CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F6 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 2% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 4·8' H/D = 0·96
Tailwater T : 1·6'
Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 1·06'
Add for approach velocity + 0·02'
Corrected head 1·08' Q = 96 c.f.s.
Tailwater 0·5' above crest : correction factor 0·86
$Q/D^{5/2}$ = 1·71 Critical depth d_c = 2·7'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	3·8	2·8	2·5	2·2	1·95	1·95	1·5	1·25
	Calc'd depth	3·55	2·75	2·65	2·55	2·5	2·7	2·45	2·4

SPECIAL OBSERVATIONS : Piezometer readings believed to be inaccurate.

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2·7'	11·32	1·22	8·48	0·141	0·030

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2·75	2·05	1·03	2·0

FLOW PROFILE :

ASSUMED UNIFORM, $d \doteq 2{\cdot}7'$. (CRITICAL)

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F7 LENGTH : 72' net INLET Bevel projecting
DIA. : 60" SLOPE : 2% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 5·5' H/D = 1·1
Tailwater T : 1·6'
Inlet d_e : Not read Outlet d_o = 3·1'

DISCHARGE : Observed weir head h' = 1·19'
Add for approach velocity + 0·02
Corrected head 1·21' Q = 118 c.f.s.
Tailwater 0·5' above crest: correction factor 0·89
$Q/D^{5/2}$ = 2·11 Critical depth d_c = 3·0'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	4·25	3·05	2·90	2·60	2·4	2·45	2·0	1·6
	Calc'd depth	4·0	3·0	3·05	2·95	2·95	3·2	2·95	2·75

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
3·0	13·0	1·26	9·1	0·141	0·029

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3·0'	2·5'	1·28	1·95

FLOW PROFILE :

Assumed uniform critical, d = 3·0'.

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F8 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 5.2' H/D = 1.04
Tailwater T : 2.4'
Inlet d_e : Not read Outlet d_o = 2.8'

DISCHARGE : Observed weir head h = 1.02
Add for approach velocity + 0.02
Corrected head 1.04 Q = 106 c.f.s.

$Q/D^{5/2}$ = 1.89 Critical depth d_c = 2.8'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	4.0	2.9	2.55	2.1	1.6	1.25	0.65	0.55
	Calc'd depth	3.6	2.85	2.85	2.7	2.55	2.5	2.25	2.5

SPECIAL OBSERVATIONS : Piezometer readings near outlet incorrect. Visual observation and photographs indicated uniform depth through culvert.

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2.8'	12.0	1.23	8.8	0.181	0.038

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2.8	2.4	1.21	2.0

FLOW PROFILE :

Uniform critical, d = 2.8'.

CULVERT FIELD TESTS — DATA & ANALYSIS

TEST Nº : F9   LENGTH : 72′ net   INLET : Bevel projecting

DIA. : 60″   SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 6.4′   H/D = 1.28

Tailwater T : 2.3′

Inlet d_e : Not read   Outlet d_o = 3.2′

DISCHARGE :

Observed weir head h = 1.20

Add for approach velocity + 0.03

Corrected head 1.23   Q = 136 c.f.s.

$Q/D^{5/2}$ = 2.43   Critical depth d_c = 3.2′

PIEZOMETERS

Nº.	1	2	3	4	5	6	7	8
Reading	4.55	3.15	3.05	2.65	2.15	1.75	1.15	0.90
Calc'd depth		3.1	3.3	3.2	3.1	3.0	2.75	2.8

SPECIAL OBSERVATIONS : Flow surface highly aerated. Visual observation indicated uniform depth through culvert.

BACKWATER CALCULATION : Assumed n =

S_f = $/(AR^{2/3})^2$

$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f′	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
3.2′	14.0	1.29	9.7	0.181	0.036

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3.2	3.2	1.46	2.2

FLOW PROFILE :

Assumed uniform critical, d = 3.2′.

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F10 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 7.2' H/D = 1.44
Tailwater T : 2.4'
Inlet d_e : Not read Outlet d_o = 3.5'.

DISCHARGE : Observed weir head h = 1.30'
Add for approach velocity + 0.03
Corrected head 1.33 Q = 153 c.f.s.

$Q/D^{5/2}$ = 2.74 Critical depth d_c = 3.5'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	4.95	3.45	3.4	2.9	2.45	2.1	1.45	1.1
Calc'd depth		3.4	3.7	3.5	3.4	3.4	3.05	3.05

SPECIAL OBSERVATIONS : Discount piezometers 7 & 8.
Flow highly aerated.

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
3.5'	15.7	1.31	9.75	0.181	0.036

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3.5'	3.7'	1.48	2.5

FLOW PROFILE :

Uniform critical, d = 3.5'

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F11 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 3.8' H/D = 0.76
Tailwater T : 2.3'
Inlet d_e : 2.0' , Outlet d_o = 2.2'

DISCHARGE : Observed weir head h = 0.68
Add for approach velocity + 0.01
Corrected head 0.69 Q = 58 c.f.s.

$Q/D^{5/2}$ = 1.04 Critical depth d_c = 2.05'

PIEZOMETERS Nº	1	2	3	4	5	6	7	8
Reading	3.0	2.15	1.75	1.35	0.9	0.55	0.4	0.2
Calc'd depth		2.1	2.05	1.95	1.85	1.8	2.0	2.15

SPECIAL OBSERVATIONS : Visual observation indicated nearly uniform flow through culvert

BACKWATER CALCULATION : Assumed n = 0.035 $S_f = 1.59/(AR^{2/3})^2$
$1.3\,V^2/2g = 68.0/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
2.0	7.9	1.07	1.09	3.09					
1.9	7.3	1.04	1.27	3.17	0.08	0.0276	.0054	15'	
1.8	6.8	1.02	1.47	3.27	0.10	.0331	0	∞	

Normal depth = 1.8'

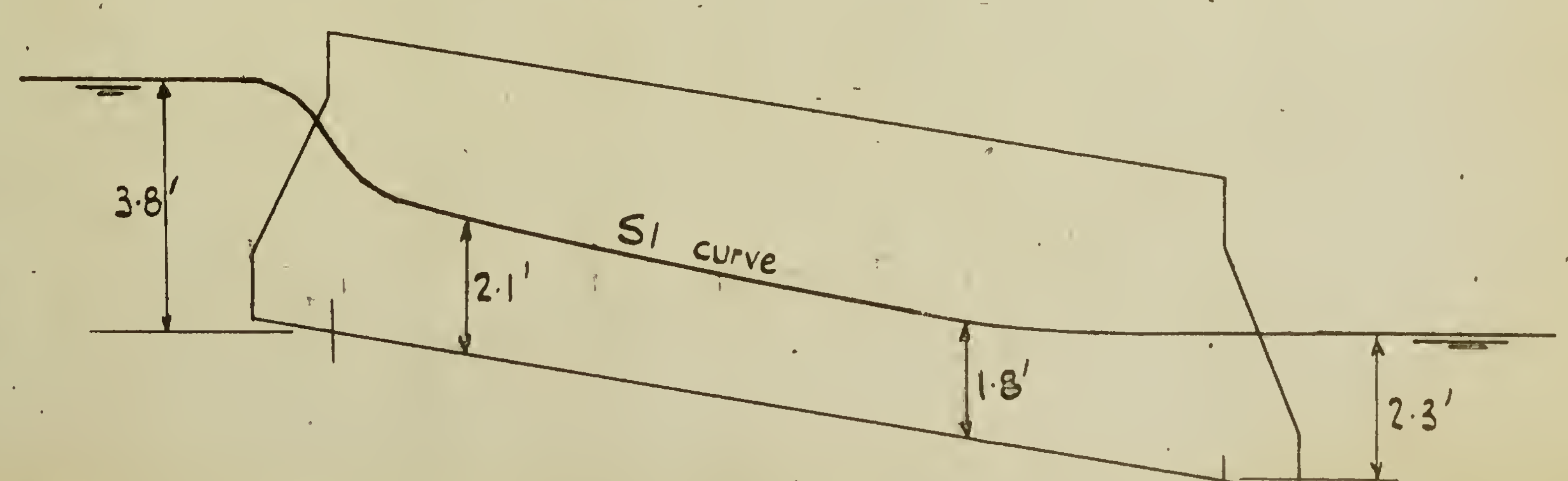
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2.2	1.6	0.68	2.35

FLOW PROFILE : (ASSUMED)



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F 12 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 3.3% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 3.1' H/D = 0.62
Tailwater T : 2.2'
Inlet d_e : Not read Outlet d_o = 2.2'

DISCHARGE : Observed weir head h = 0.50
Add for approach velocity + ⁄
Corrected head Q = 36 c.f.s.

$Q/D^{5/2}$ = 0.64 Critical depth d_c = 1.6'

PIEZOMETERS :

Nº	1	2	3	4	5	6	7	8
Reading			Not	read				
Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
	No	data	

FLOW PROFILE : No data.

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F13 LENGTH : 72' net INLET : Bevel projecting.
DIA. : 60" SLOPE : ~1% $D^{5/2}$ = 56.

OBSERVED DEPTHS : Headwater H : 3.3' H/D = 0.66
Tailwater T : 0.5'
Inlet d_e : 2.2' Outlet d_o = 2.2'

DISCHARGE : Observed weir head h = 0.63
Add for approach velocity + —
Corrected head — Q = 50 c.f.s.

$Q/D^{5/2}$ = 0.89 Critical depth d_c = 1.9'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	2.7	2.3	2.3	2.2	2.1	2.0	1.9	1.6
	Calc'd depth	2.55	2.25	2.35	2.35	2.35	2.35	2.35	2.15

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :	d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
	2.35	9.6	1.15	5.21	0.10	0.033

ENTRANCE DRAWDOWN CALCULATION :	d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
	2.35	0.95	0.42	2.25

FLOW PROFILE : Uniform sub-critical, d = 2.35'.

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F14 LENGTH : 72' net INLET : Bevel projecting

DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 4·8' H/D = 0·96

Tailwater T : 0·7'

Inlet d_e : 3·6' Outlet d_o = 2·9'

DISCHARGE : Observed weir head h = 0·95'

Add for approach velocity + ·02

Corrected head 0·97 Q = 95 c.f.s.

$Q/D^{5/2}$ = 1·70 Critical depth d_c = 2·7'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	3·9	3·65	3·45	3·25	3·1	2·9	2·75	2·3
Calc'd depth	3·75	3·6	3·5	3·4	3·35	3·25	3·2	2·85

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = 0·037 $S_f = 5{\cdot}59/(AR^{2/3})^2$

$1{\cdot}3V^2/2g = 182/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
2·9	12·5	1·25	1·17	4·07					
3·1	13·5	1·28	1·00	4·10	0·03	·0187	·0087	3'	3'
3·3	14·5	1·30	·86	4·16	·06	·0157	·0057	11'	14'
3·4	15·0	1·30	·81	4·21	·05	·0145	·0045	11'	25'
3·5	15·4	1·31	·77	4·27	·06	·0137	·0037	16'	41'
3·6	15·7	1·32	·74	4·34	·07	·0129	·0029	24'	65'

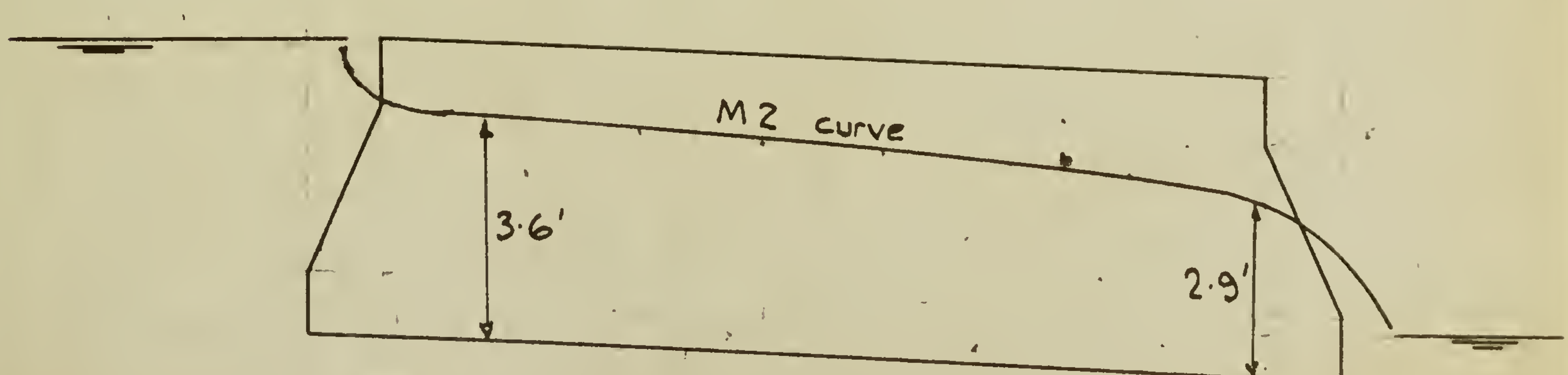
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3·6	1·2	0·57	2·1

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F15 LENGTH : 72' net INLET : Bevel projecting

DIA. : 60" SLOPE : 1% $D^{5/2}$ = ·56

OBSERVED DEPTHS : Headwater H : 6·4' H/D = 1·28

Tailwater T : 0·9'

Inlet d_e : Not readable Outlet d_o = 3·3'

DISCHARGE : Observed weir head h = 1·17

Add for approach velocity + 0·03

Corrected head 1·20 Q = 131 c.f.s.

$Q/D^{5/2}$ = 2·34 Critical depth d_c = 3·2'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	5·4	4.45	4·25	4·05	3·85	3·55	3·3	2·7
	Calc'd depth		4·4	4·3	4·2	4·1	3·9	3·75	3·25

SPECIAL OBSERVATIONS : Strong vortex at inlet

BACKWATER CALCULATION : Assumed n = 0·034 $S_f = 8\cdot95/(AR^{2/3})^2$

$1\cdot3\, V^2/2g = 347/A^2$

d	A	$R^{2/3}$	$1\cdot3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3·3	14·5	1·30	1·65	4·95					
3·7	16·2	1·32	1·33	5·03	0·08	·0194	·0094	9	9
3·9	17·0	1·32	1·20	5·10	0·07	·0178	·0078	9	18
4·1	17·8	1·32	1·09	5·19	0·09	·0162	·0062	14	32
4·2	18·2	1·31	1·05	5·25	·06	·0156	·0056	11	43
4·3	18·6	1·30	1·00	5·30	·05	·0152	·0052	10	53
4·4	19·0	1·29	0·96	5·36	·06	·0149	·0049	12	65

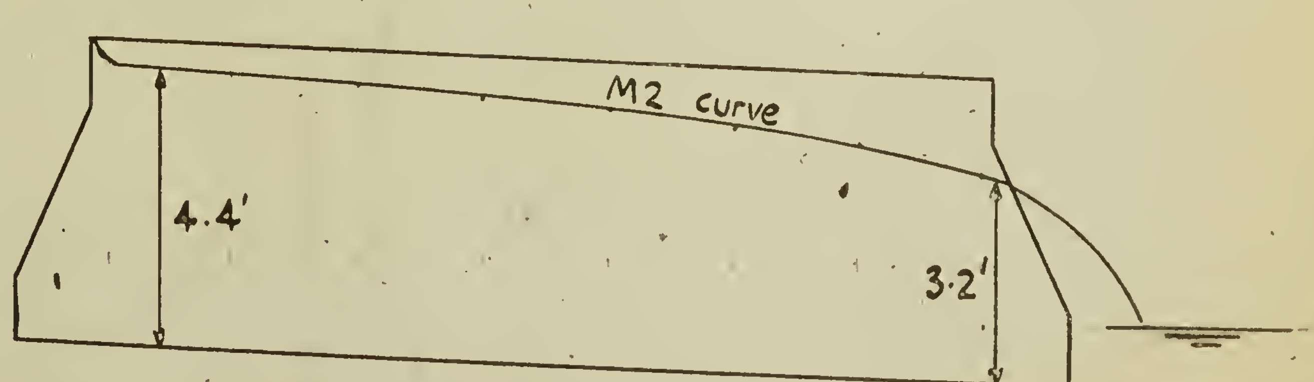
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
4·4	2·0	0·74	2·7

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F16 LENGTH : 72' net INLET : Bevel projecting
DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 7.2' H/D = 1.44
Tailwater T : 0.9'
Inlet d_e : Not read Outlet d_o = 3.5'

DISCHARGE : Observed weir head h = 1.23
Add for approach velocity + .03
Corrected head 1.26 Q = 141 c.f.s.

$Q/D^{5/2}$ = 2.52 Critical depth d_c = 3.3'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	5.9	4.9	4.6	4.3	4.1	3.8	3.5	2.8
Calc'd depth		4.9	4.7	4.5	4.4	4.2	4.0	3.4

SPECIAL OBSERVATIONS : 12" dia. continuous vortex at inlet

BACKWATER CALCULATION : Assumed n = 0.036 $S_f = 11.6/(AR^{2/3})^2$
$1.3\,V^2/2g = 401/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3.5	15.4	1.31	1.69	5.19					
4.0	17.4	1.32	1.33	5.33	0.14	.0219	.0119	12	12
4.2	18.2	1.31	1.21	5.41	.08	.0203	.0103	8	20
4.4	19.0	1.29	1.10	5.50	.09	.0193	.0093	10	30
4.6	19.4	1.26	1.07	5.67	.17	.0194	.0094	18	48
4.8	19.6	1.16	1.04	5.84	.17	.0223	.0123	14	62

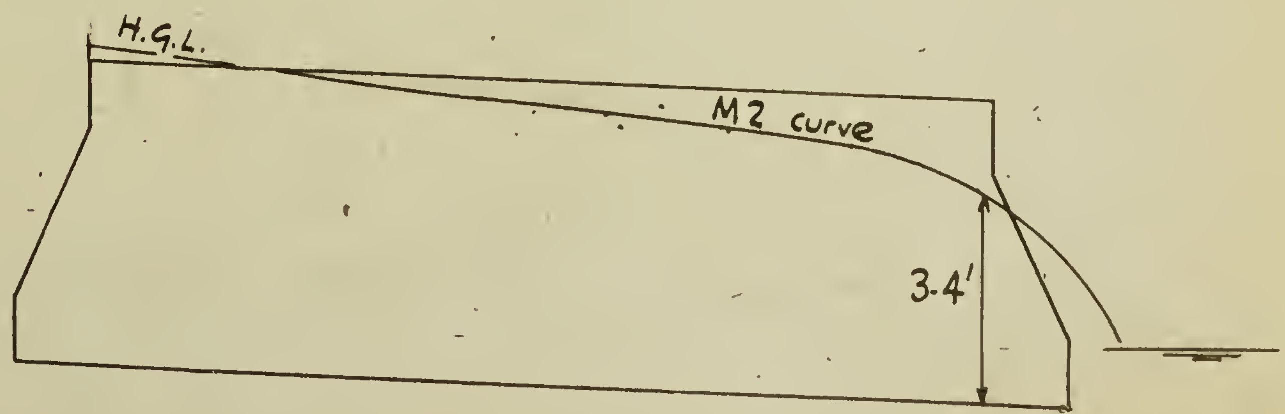
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
5.0	2.2	0.80	2.75

~~FLOW PROFILE~~



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F17 LENGTH : 72′ net INLET : Bevel projecting
DIA. : 60″ SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 7·6′ , H/D = 1·52
Tailwater T : 1·0′
Inlet d_e : Not read Outlet d_o = 3·5′

DISCHARGE : Observed weir head h = 1·27′
Add for approach velocity + 0·03
Corrected head 1·30 Q = 148 c.f.s.

$Q/D^{5/2}$ = 2·64 Critical depth d_c = 3·5′

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	6·2	5·1	4·8	4·5	4·2	3·8	3·5	2·9
Calc'd depth		5·1	4·9	4·7	4·5	4·2	4·0	3·5

SPECIAL OBSERVATIONS : Continuous vortex at inlet

BACKWATER CALCULATION : Assumed n = ·036 $S_f = 12.8/(AR^{2/3})^2$
$V^2/2g = 448/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3·5	15·4	1·31	1·88	5·38					
4·0	17·4	1·32	1·48	5·48	0·10	·0242	·0142	7	7
4·2	18·2	1·31	1·35	5·55	·07	·0226	·0126	6	13
4·4	19·0	1·29	1·23	5·63	·08	·0204	·0104	8	21
4·6	19·4	1·26	1·18	5·78	·15	·0205	·0105	14	35
4·8	19·6	1·16	1·16	5·96	·18	·0247	·0147	12	47

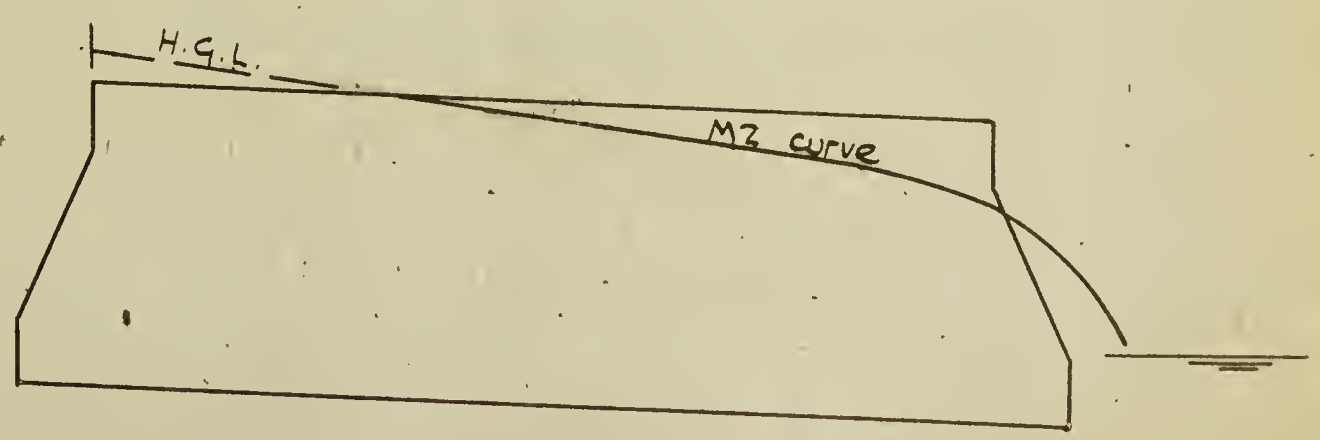
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
5·25	2·35	0·89	2·64

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F18   LENGTH : 72' net   INLET : Square projecting

DIA. : 60"   SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 2·9'   H/D = 0·58

Tailwater T : 0·5'

Inlet d_e : 2·3'   Outlet d_o = 2·0'

DISCHARGE : Observed weir head h = 0·60

Add for approach velocity + 0·01

Corrected head 0·61   Q = 48 c.f.s.

$Q/D^{5/2}$ = 0·86   Critical depth d_c = 1·9'

PIEZOMETERS								
Nº	1	2	3	4	5	6	7	8
Reading	2·4	2·45	2·25	2·15	2·0	1·9	1·8	1·5
Calc'd depth	2·45	2·4	2·3	2·3	2·25	2·25	2·25	2·05

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$

$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2·3	9·3	1·14	5·16	0·10	0·033

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2·3	0·6	0·42	1·5

FLOW PROFILE :

Approx. uniform sub-critical, d = 2·3'

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F19 LENGTH : 72' net INLET : Square projecting
DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 4.25' H/D = 0.85
Tailwater T : 0.8'
Inlet d_e : 3.4' Outlet d_o = 2.9'

DISCHARGE : Observed weir head h = 0.91
Add for approach velocity + 0.01
Corrected head 0.92 Q = 88 c.f.s.

$Q/D^{5/2}$ = 1.57 Critical depth d_c = 2.5'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	3.2	3.4	3.25	3.13	2.97	2.8	2.65	2.2
	Calc'd depth	3.25	3.35	3.3	3.3	3.2	3.15	3.1	2.85

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
3.3'	14.5	1.30	6.06	0.10	0.032

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3.3	0.95	0.57	1.67

FLOW PROFILE :

Uniform sub-critical, d = 3.3', in upstream half.

CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F21' LENGTH : 72' net INLET : Square projecting
DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 5·6' H/D = 1·12
Tailwater T : 0·8'
Inlet d_e : Not read Outlet d_o = 3·2'

DISCHARGE : Observed weir head h = 1·13'
Add for approach velocity + 0·02
Corrected head 1·15 Q = 123 c.f.s.

$Q/D^{5/2}$ = 2·20 Critical depth d_c = 3·1'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	3·6	4·3	4·1	3·95	3·75	3·5	3·2	2·65
Calc'd depth		4·25	4·15	4·1	4·0	3·85	3·65	3·2

SPECIAL OBSERVATIONS : Vortex at inlet

BACKWATER CALCULATION : Assumed n = 0·035 $S_f = 8.37/(AR^{2/3})^2$
$1.3V^2/2g = 306/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3·2	14·0	1·29	1·56	4·76					
3·6	15·7	1·32	1·23	4·83	0·07	·0195	·0095	7	7
3·8	16·6	1·32	1·10	4·90	0·07	·0175	·0075	9	16
4·0	17·4	1·32	1·01	5·01	·11	·0158	·0058	19	35
4·1	17·8	1·32	·96	5·06	·05	·0152	·0052	10	45
4·2	18·2	1·31	·92	5·12	·06	·0147	·0047	13	58

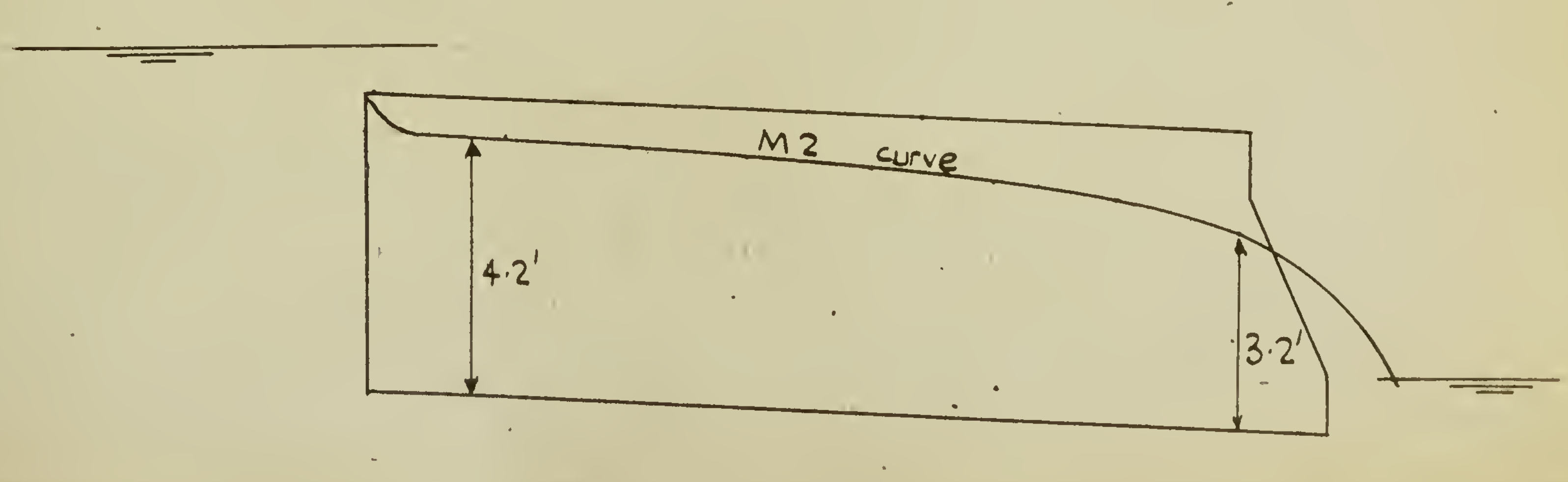
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
4·2	1·4	0·71	2·0

FLOW PROFILE :



CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F22 LENGTH : 72' net INLET : Square projecting
DIA. : 60" SLOPE : 0 $D^{5/2}$ = 56

OBSERVED DEPTHS :
Headwater H : 3.3' $H/D = 0.66$
Tailwater T : —
Inlet d_e : 3.0' Outlet $d_o = 2.1'$

DISCHARGE :
Observed weir head h = 0.60
Add for approach velocity + ·01
Corrected head ·61 Q = 48 c.f.s.

$Q/D^{5/2} = 0.86$ Critical depth $d_c = 1.9'$.

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	2.85	3.0	2.9	2.8	2.7	2.6	2.5	2.2
Calc'd depth		2.95	2.85	2.75	2.65	2.55	2.45	2.15

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION :
Assumed n = 0.036 $S_f = 1.34/(AR^{2/3})^2$
$1.3\,V^2/2g = 46.5/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
2.1	8.25	1.10	0.69	2.79					
2.4	9.83	1.16	0.48	2.88	0.09		·0104	9	9
2.6	11.0	1.20	0.39	2.99	·13		·0077	17	26
2.8	12.0	1.23	·32	3.12	·13		·0063	21	47
2.9	12.5	1.25	·30	3.20	·08		·0056	14	61
3.0	13.0	1.26	·27	3.27	·07		·0050	14	75

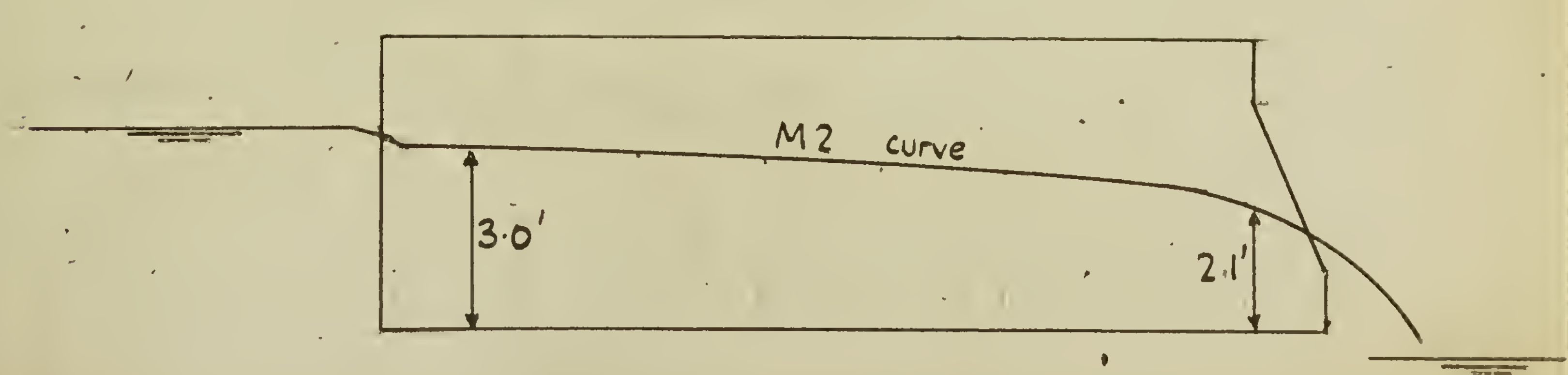
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3.0	0.3	0.21	1.5

FLOW PROFILE :



CULVERT FIELD TESTS — DATA & ANALYSIS

TEST Nº : F23 LENGTH : 72' net INLET : Square projecting
DIA. : 60" SLOPE : 0 $D^{5/2}$ = 56.

OBSERVED DEPTHS : Headwater H : 4·25' H/D = 0·85
Tailwater T : —
Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 0·82
Add for approach velocity + ·01
Corrected head 0·83 Q = 76 c.f.s.

$Q/D^{5/2}$ = 1·36 Critical depth d_c = 2·4'

PIEZOMETERS Nº	1	2	3	4	5	6	7	8
Reading	3·6	3·75	3·6	3·5	3·4	3·3	3·1	2·7
Calc'd depth	3·55	3·7	3·55	3·45	3·35	3·25	3·05	2·65

SPECIAL OBSERVATIONS : Smooth inlet conditions, in contrast to bevel inlet.

BACKWATER CALCULATION : Assumed n = 0·035 $S_f = 3{\cdot}2\,/(AR^{2/3})^2$
$1{\cdot}3\,V^2/2g = 116{\cdot}4/A^2$

d	A	$R^{2/3}$	$1{\cdot}3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
2·6	11·0	1·20	0·96	3·56					
3·0	13·0	1·26	·69	3·69	0·13		·0120	11	11
3·2	14·0	1·29	·60	3·80	·11		·0098	11	22
3·4	15·0	1·30	·52	3·92	·12		·0084	14	36
3·6	15·7	1·32	·47	4·07	·15		·0074	20	56
3·7	16·2	1·32	·44	4·14	·07		·0070	10	66

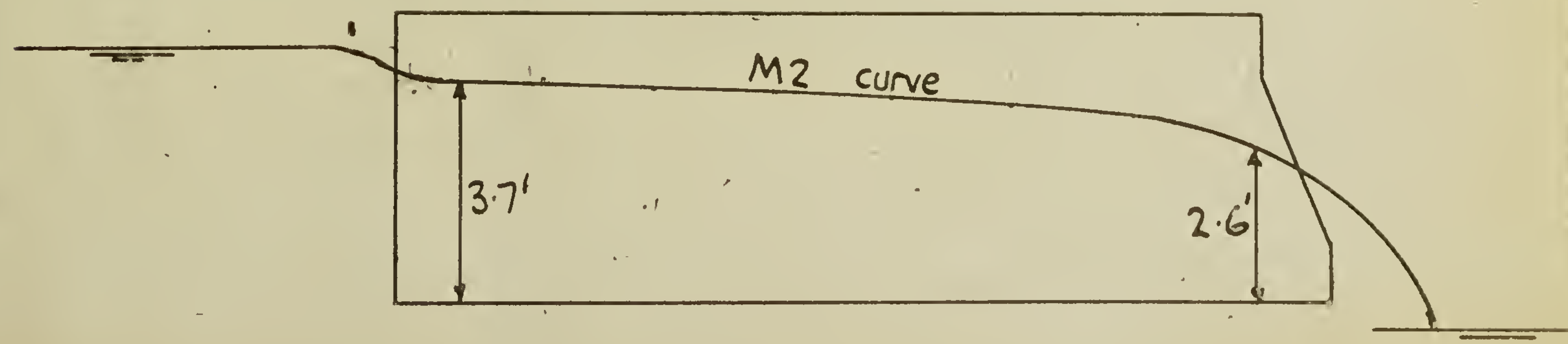
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1{\cdot}49\,R^{2/3}S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3·7	0·55	0·34	1·62

FLOW PROFILE :



CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F24 LENGTH : 72' net INLET Square projecting
DIA. : 60" SLOPE : 0 $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 6·0' H/D = 1·20
Tailwater T : 0·3'
Inlet d_e : Not read Outlet d_o = 3·2'

DISCHARGE : Observed weir head h = 1·10'
Add for approach velocity + 0·02
Corrected head 1·12 Q = 118 c.f.s.

$Q/D^{5/2}$ = 2·11 Critical depth d_c = 3·0'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading		4·85	4·65	4·45	4·25	4·05	3·8	3·25
Calc'd depth		4·8	4·6	4·4	4·2	4·0	3·75	3·2

SPECIAL OBSERVATIONS : Strong vortex at inlet

BACKWATER CALCULATION : Assumed n = 0·036 $S_f = 8{\cdot}15/(AR^{2/3})^2$
$1{\cdot}3V^2/2g = 281/A^2$

d	A	$R^{2/3}$	$1{\cdot}3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3·2	14·0	1·29	1·43	4·63					
3·7	16·2	1·32	1·07	4·77	0·14		·0178	8	8
4·0	17·4	1·32	·92	4·92	·15		·0155	10	18
4·3	18·6	1·30	·81	5·11	·19		·0139	14	32
4·6	19·4	1·26	·74	5·34	·23		·0136	17	51
4·8	19·6	1·16	·73	5·53	·19		·0159	12	63

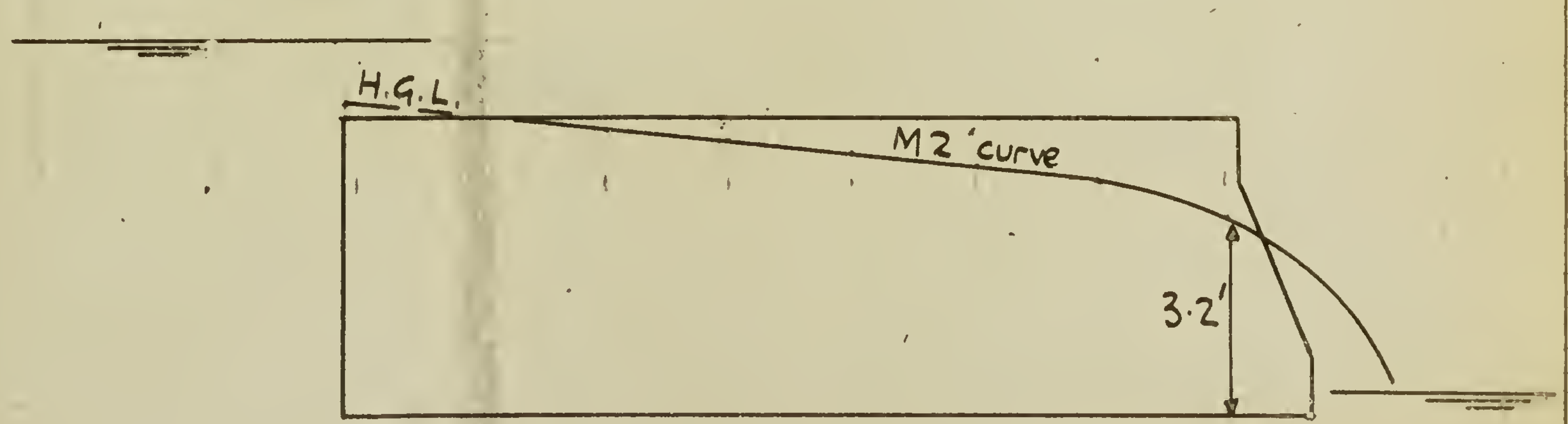
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1{\cdot}49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
5·0	1·0	0·56	1·8

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F25 LENGTH : 72' net INLET : Square projecting
DIA. : 60" SLOPE : 0 $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 7·2' H/D = 1·44
Tailwater T : 0·4'
Inlet d_e : Full Outlet d_o = 3·5'

DISCHARGE : Observed weir head h = 1·21
Add for approach velocity + ·02
Corrected head 1·23 Q = 137 c.f.s.

$Q/D^{5/2}$ = 2·45 Critical depth d_c = 3·2'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading		5·45	5·2	4·95	4·7	4·4	4·1	3·5
	Calc'd depth		5·4	5·15	4·9	4·65	4·35	4·05	3·45

SPECIAL OBSERVATIONS : Intermittent weak vortex at inlet.

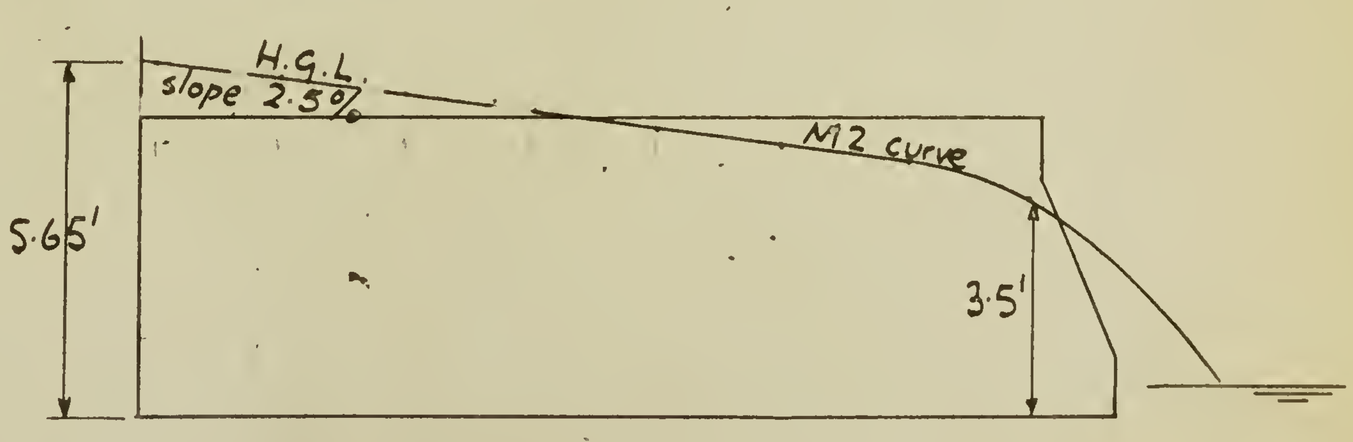
BACKWATER CALCULATION : Assumed n = 0·037 $S_f = 11·6/(AR^{2/3})^2$
$1·3V^2/2g = 380/A^2$

d	A	$R^{2/3}$	$1·3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3·5	15·4		1·60	5·10					
4·1	17·8	1·32	1·20	5·30	0·20		·0210	10	10
4·4	19·0	1·29	1·05	5·45	·15		·0193	8	18
4·7	19·5	1·22	1·00	5·70	·25		·0205	12	30
4·8	19·6	1·16	·99	5·79	·09		·0225	4	34

~~UNIFORM~~ FULL FLOW CALCULATION :	d	A	$R^{2/3}$	V	$S_f^{1/2}$	$n = \frac{1·49 R^{2/3} S^{1/2}}{V}$
	4·8	19·6	1·16	6·99	0·158	0·039

ENTRANCE DRAWDOWN CALCULATION :	d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
	5·65	1·55	0·76	2·04

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F26 LENGTH : 72′ net INLET : Square projecting
DIA. : 60″ SLOPE : 0 $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 7·9′ H/D = 1·58
Tailwater T : 0·5′
Inlet d_e : Full Outlet d_o = not read

DISCHARGE : Observed weir head h = 1·27′
Add for approach velocity + ·03
Corrected head 1·30 Q = 148 c.f.s.

$Q/D^{5/2}$ = 2·64 Critical depth d_c = 3·4′

PIEZOMETERS :

Nº	1	2	3	4	5	6	7	8
Reading	4·9	5·8	5·5	5·2	4·9	4·6	4·3	3·6
Calc'd depth		5·75	5·45	5·15	4·85	4·55	4·25	3·55

SPECIAL OBSERVATIONS : Pipe appeared full for about 3/4 length.

BACKWATER CALCULATION : Assumed n = 0·036 $S_f = 12.8/(AR^{2/3})^2$
$1.3V^2/2g = 442/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3·6	15·7		1·78	5·38					
4·1	17·8	1·32	1·39	5·49	0·11		·0232	5	5
4·5	19·2	1·28	1·20	5·70	·21		·0211	10	15
4·8	19·6	1·16	1·14	5·94	·24		·0247	10	25

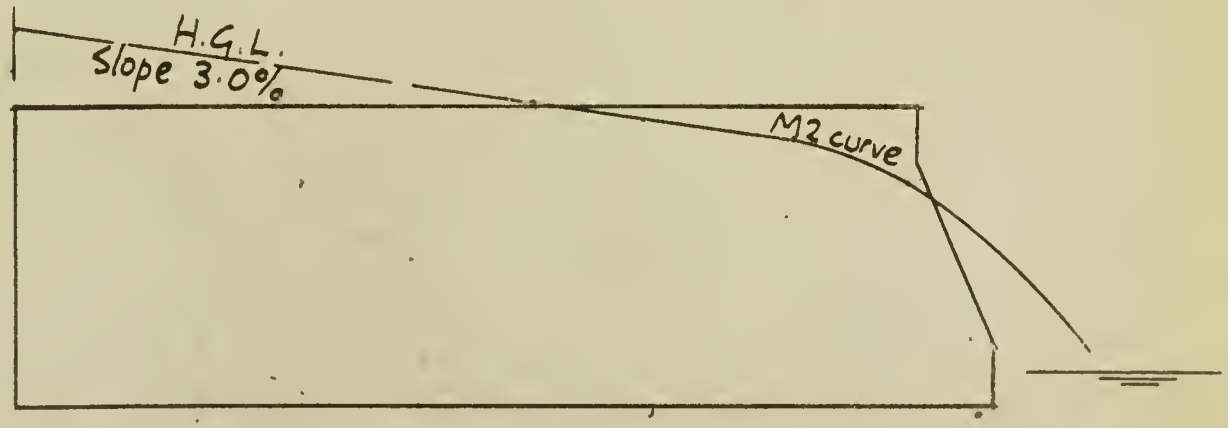
FULL ~~UNIFORM~~ FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S_f^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
4·8	19·6	1·16	7·55	0·173	0·040

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
6·05	1·85	0·88	2·1

FLOW PROFILE :



CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F27 LENGTH : 72′ net INLET : Hood
DIA. : 60″ SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 3.5′ H/D = 0.70
Tailwater T : 0.6′
Inlet d_e : 2.9′ Outlet d_o = 2.3′

DISCHARGE : Observed weir head h = 0.69′
Add for approach velocity + 0.01
Corrected head 0.70′ Q = 59 c.f.s.

$Q/D^{5/2}$ = 1.05 Critical depth d_c = 2.1′

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	3.1	2.8	2.65	2.5	2.35	2.2	2.05	1.75
Calc'd depth	2.95	2.75	2.7	2.65	2.6	2.55	2.5	2.3

SPECIAL OBSERVATIONS : Smooth inlet conditions

BACKWATER CALCULATION : Assumed n = 0.038 $S_f = 2.27/(AR^{2/3})^2$
$1.3\,V^2/2g = 70/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
2.3	9.4		0.79	3.09					
2.5	10.5	1.18	0.64	3.14	.05	.0147	.0047	11	11
2.6	11.0	1.20	.57	3.17	.03	.0129	.0029	10	21
2.7	11.5	1.21	.53	3.23	.06	.0115	.0015	40	61
2.8	12.0	1.23	.49	3.29	.06	.0104	.0004	150	

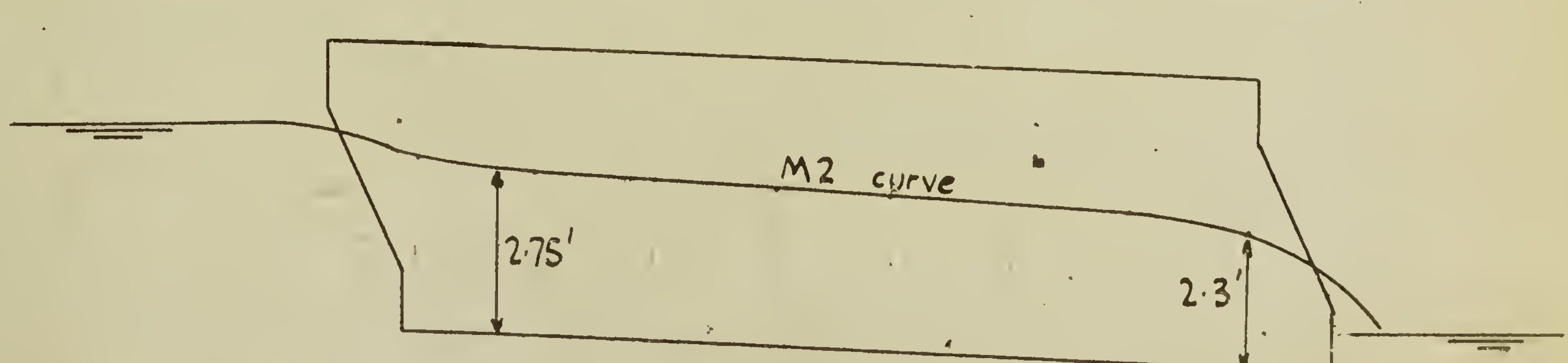
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49\,R^{2/3}S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2.8	0.7	0.38	1.84

FLOW PROFILE :



CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F28 LENGTH : 72' net INLET : Hood

DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 5·5' H/D = 1·1

Tailwater T : 0·8'

Inlet d_e : Not read Outlet d_o = 3·1'

DISCHARGE : Observed weir head h = 1·05'

Add for approach velocity + ·02

Corrected head 1·07 Q = 111 c.f.s.

$Q/D^{5/2}$ = 1·98 Critical depth d_c = 2·9'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	4·5	4·1	3·9	3·7	3·5	3·3	3·05	2·55
	Calc'd depth	4·35	4·05	3·95	3·85	3·75	3·65	3·5	3·1

SPECIAL OBSERVATIONS : No vortex at inlet

BACKWATER CALCULATION : Assumed n = 0·038 $S_f = 8{\cdot}03/(AR^{2/3})^2$

$1{\cdot}3\,V^2/2g = 250/A^2$

d	A	$R^{2/3}$	$1{\cdot}3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3·1	13·5		1·36	4·46					
3·4	15·0	1·30	1·10	4·50	0·04	·0208	·0108	4	4
3·6	15·7	1·32	1·01	4·61	·11	·0187	·0087	13	17
3·8	16·6	1·32	·91	4·71	·10	·0167	·0067	15	32
4·0	17·4	1·32	·82	4·82	·11	·0152	·0052	21	53
4·1	17·8	1·32	·79	4·89	·07	·0145	·0045	16	69

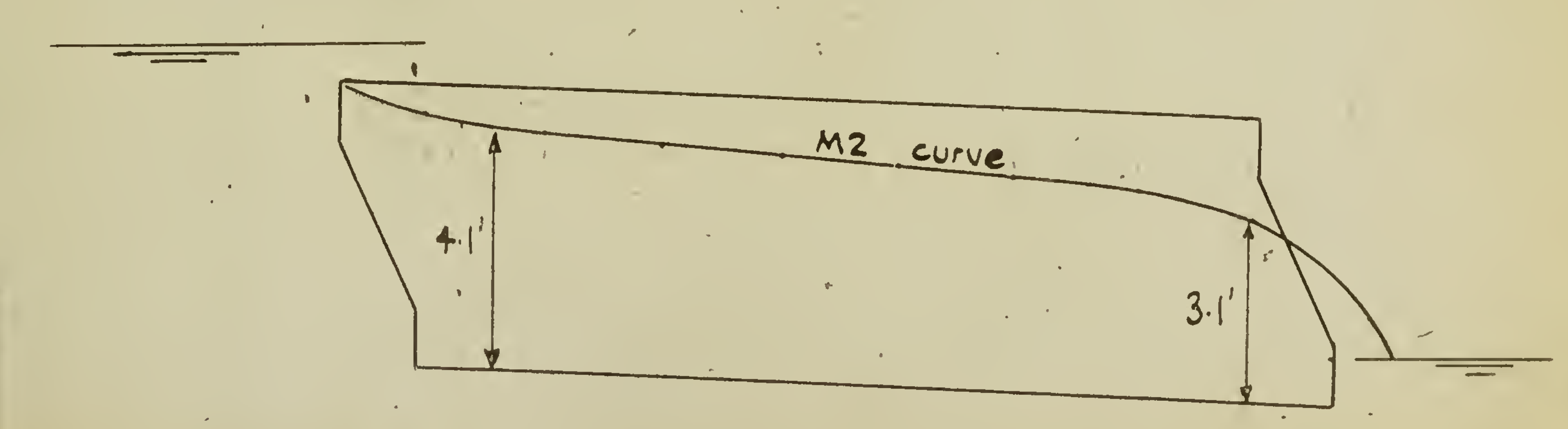
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \dfrac{1{\cdot}49\,R^{2/3}S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
4·1	1·4	0·61	2·3

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F29 LENGTH : 72' net INLET Hood
DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 6.4' H/D = 1.28
Tailwater T : 0.8'
Inlet d_e : Not read Outlet d_o = 3.3'

DISCHARGE : Observed weir head h = 1.14
Add for approach velocity + .02
Corrected head 1.16 Q = 125 c.f.s.

$Q/D^{5/2}$ = 2.24 Critical depth d_c = 3.1'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	Fluctuating	4.65	4.4	4.15	3.9	3.6	3.3	2.75
Calc'd depth		4.6	4.45	4.3	4.15	3.95	3.75	3.3

SPECIAL OBSERVATIONS : No vortex

BACKWATER CALCULATION : Assumed n = 0.039 $S_f = 10.7/(AR^{2/3})^2$
$1.3V^2/2g = 316/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3.3	14.5		1.49	4.79					
3.7	16.2	1.32	1.20	4.90	0.11	.0234	.0134	8	8
4.0	17.4	1.32	1.04	5.04	.14	.0202	.0102	14	22
4.2	18.2	1.31	.95	5.15	.11	.0188	.0088	12	34
4.4	19.0	1.29	.87	5.27	.12	.0178	.0078	15	49
4.6	19.4	1.26	.83	5.43	.16	.0178	.0078	21	70

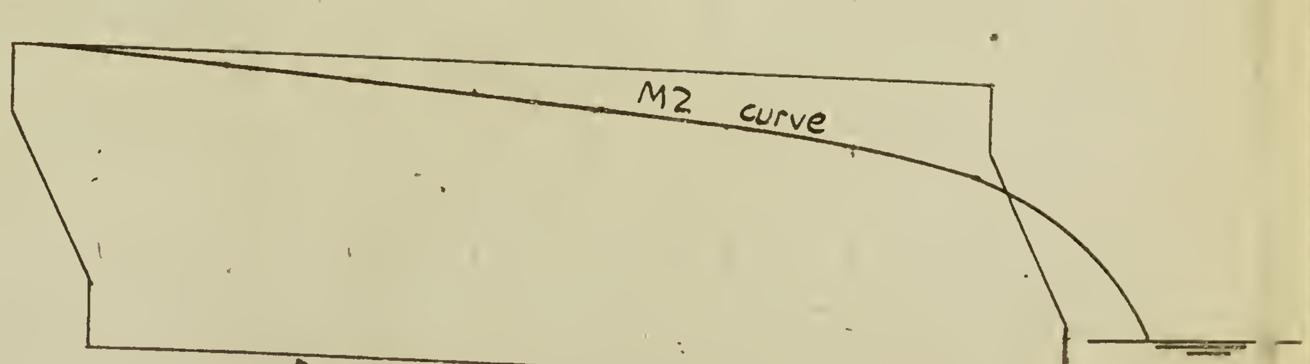
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
4.8	1.6	0.64	2.5

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F30 LENGTH : 72' net INLET : Hood
DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 7.5' H/D = 1.50
Tailwater T : 0.9'
Inlet d_e : Full Outlet d_o = 3.5'

DISCHARGE : Observed weir head h = 1.23'
Add for approach velocity + .03
Corrected head 1.26 Q = 141 c.f.s.

$Q/D^{5/2}$ = 2.52 Critical depth d_c = 3.3'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading		5.2	4.8	4.5	4.2	3.85	3.5	2.9
Calc'd depth		5.15	4.85	4.65	4.45	4.2	3.95	3.45

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = 0.038 $S_f = 12.95/(AR^{2/3})^2$
$1.3V^2/2g = 401/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3.5	15.4		1.69	5.19					
3.9	17.0	1.32	1.39	5.29	0.10	.0256 ~~.0285~~	.0156	6	6
4.2	18.2	1.31	1.21	5.41	.12	.0228	.0128	9	15
4.4	19.0	1.29	1.10	5.50	.09	.0216	.0116	8	23
4.6	19.4	1.26	1.07	5.67	.17	.0217	.0117	14	37
4.8	19.6	1.16	1.04	5.84	.17	.0251	.0151	11	48

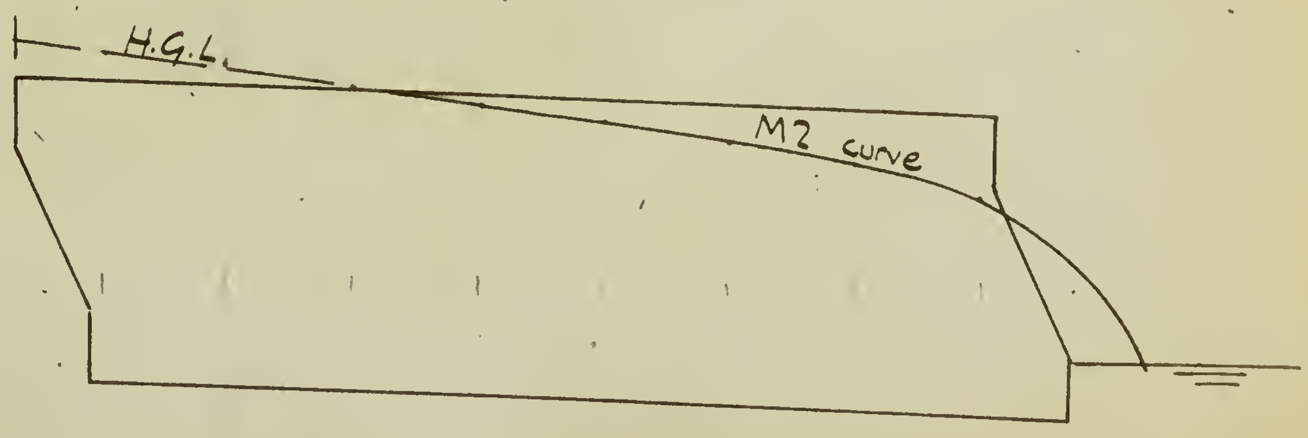
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
5.5	2.0	0.80	2.5

FLOW PROFILE :



CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F31 LENGTH : 72' net INLET : Hood

DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 8.4' H/D = 1.68

Tailwater T : 1.0'

Inlet d_e : Full Outlet d_o = 3.6'

DISCHARGE : Observed weir head h = 1.32

Add for approach velocity + .03

Corrected head 1.35 Q = 157 c.f.s.

$Q/D^{5/2}$ = 2.80 Critical depth d_c = 3.5'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	5.8	5.5	5.2	4.8	4.5	4.1	3.7	3.1
Calc'd depth	5.7	5.5	5.3	5.0	4.8	4.5	4.2	3.7

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = 0.037 $S_f = 15.2/(AR^{2/3})^2$

$1.3\,V^2/2g = 498/A^2$

d	A	$R^{2/3}$	$1.3V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3.7	16.2	1.32	1.86	5.56					
4.2	18.2	1.31	1.47	5.67	0.11	.0268	.0168	7	7
4.5	19.2	1.28	1.33	5.83	.16	.0252	.0152	11	18
4.8	19.6	1.16	1.27	6.07	.24	.0294	.0194	12	30

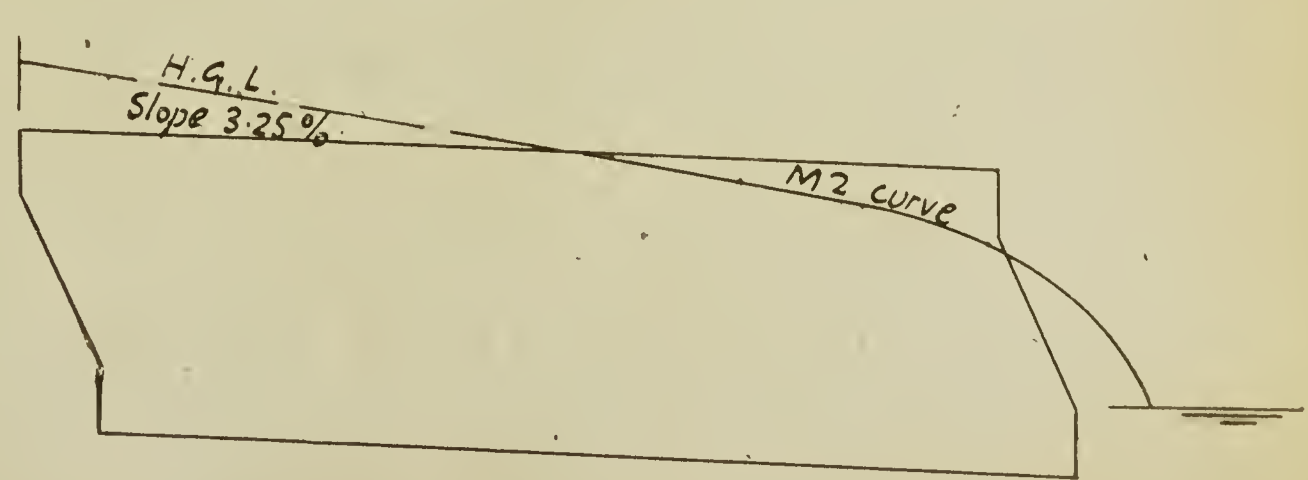
FULL ~~UNIFORM~~ FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S_f^{1/2}$	$n = \frac{1.49\,R^{2/3}S^{1/2}}{V}$
4.8	19.6	1.16	8.0	0.180	0.039

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
5.8	2.6	0.98	2.6

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F32 LENGTH : 72' net INLET : Bevel flush
DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 2.5' H/D = 0.5
Tailwater T : 0.4'
Inlet d_e : 2.0' Outlet d_o = 1.6'

DISCHARGE : Observed weir head h = 0.44
Add for approach velocity + —
Corrected head Q = 30 c.f.s.

$Q/D^{5/2}$ = 0.54 Critical depth d_c = 1.5'

PIEZOMETERS :

Nº	1	2	3	4	5	6	7	8
Reading	2.05	2.0	1.9	1.75	1.6	1.5	1.4	1.1
Calc'd depth	1.9	1.95	1.95	1.90	1.85	1.85	1.85	1.65

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
1.9'	7.23	1.04	4.15	0.10	0.037

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2.0	0.5	0.27	1.85

FLOW PROFILE :

Assumed uniform sub-critical, d = 1.9'

CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : F33 LENGTH : 72′ net INLET : Bevel flush
DIA. : 60″ SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 3·3′ H/D = 0·66
Tailwater T : 0·6′
Inlet d_e : 2·6′ Outlet d_o = 2·2′

DISCHARGE : Observed weir head h = 0·65
Add for approach velocity + ·01
Corrected head 0·66 Q = 54 c.f.s

$Q/D^{5/2}$ = 0·96 Critical depth d_c = 2·0′

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	2·7	2·65	2·5	2·35	2·2	2·1	2·0	1·65
	Calc'd depth	2·55	2·6	2·55	2·5	2·45	2·45	2·45	2·2

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2·55	10·6	1·19	5·1	0·10	0·035

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2·6	0·7	0·41	1·7

FLOW PROFILE :

Assumed uniform sub-critical, d = 2·55′

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F 34 LENGTH : 72' net INLET : Bevel flush
DIA. : 60" SLOPE : 1% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 5.4' H/D = 1.08
Tailwater T : 0.8
Inlet d_e : Not read Outlet d_o = 3.1'

DISCHARGE : Observed weir head h = 1.05
Add for approach velocity + .02
Corrected head 1.07 Q = 111 cfs.

$Q/D^{5/2}$ = 1.98 Critical depth d_c = 2.9'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading	4.4	4.05	3.85	3.65	3.45	3.25	3.05	2.5
	Calc'd depth	4.25	4.0	3.9	3.8	3.7	3.6	3.5	3.05

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = 0.037 $S_f = 7.61/(AR^{2/3})^2$
$1.3\,V^2/2g = 250/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3.1	13.5		1.36	4.46					
3.4	15.0	1.30	1.10	4.50	0.04	.0198	.0098	4	4
3.6	15.7	1.32	1.01	4.61	.11	.0177	.0077	14	18
3.8	16.6	1.32	.91	4.71	.10	.0158	.0058	17	35
4.0	17.4	1.32	.82	4.82	.11	.0144	.0044	25	60

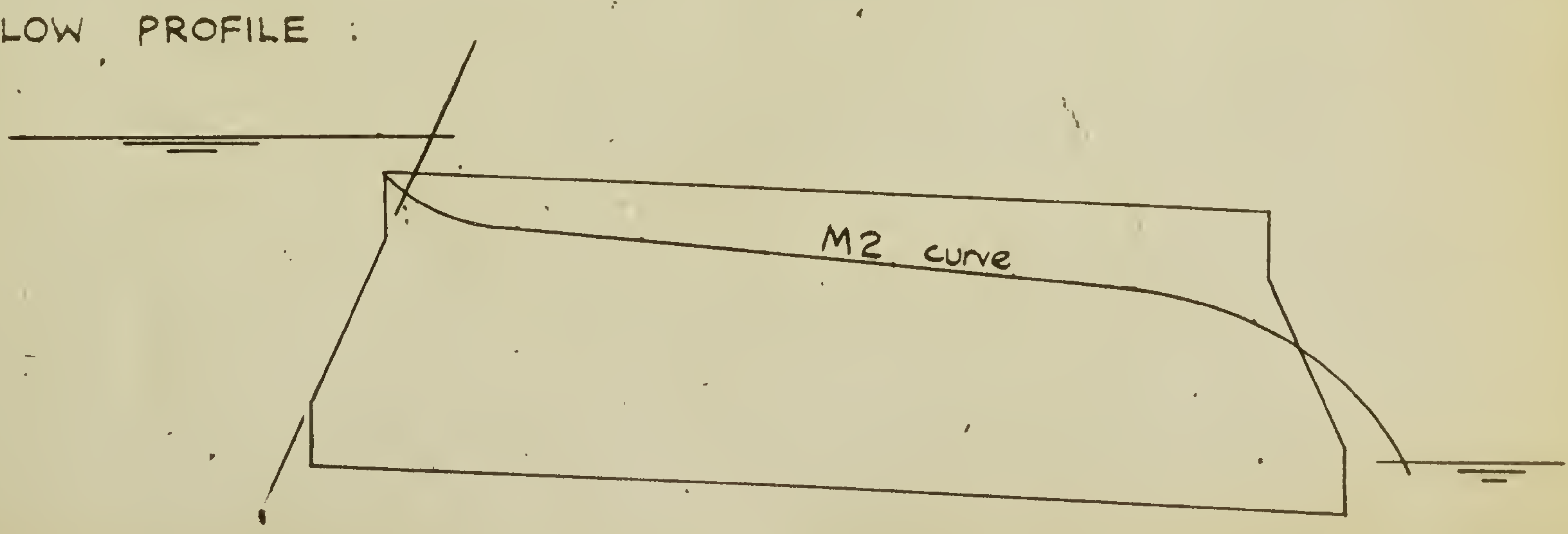
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
4.1	1.3	0.60	2.16

FLOW PROFILE :



CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : F35 LENGTH : 72' net INLET : Bevel flush
DIA. : 60" SLOPE : 2·8% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 4·0' H/D = 0·80
Tailwater T : 1·8'
Inlet d_e : 2·6' Outlet d_o = 2·4'

DISCHARGE : Observed weir head h = 0·84'
Add for approach velocity + 0·01
Corrected head 0·85 Q = 78 c.f.s.

$Q/D^{5/2}$ = 1·39 Critical depth d_c = 2·4'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading	3·1	2·7	2·35	2·0	1·55	1·15	0·95	0·7
Calc'd depth	2·75	2·65	2·6	2·5	2·35	2·2	2·3	2·3

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2.4	9·83	1·16	7·94	0·167	0·036

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2.4	1·6	0·98	1·6

FLOW PROFILE :

Assumed uniform critical, d = 2·4'.

CULVERT FIELD TESTS — DATA & ANALYSIS

TEST Nº : F36 LENGTH : 72' net INLET: Bevel flush
DIA. : 60" SLOPE : 2.8% $D^{5/2}$ = 56

OBSERVED DEPTHS : Headwater H : 4.8' H/D = 0.96
Tailwater T : 1.8'
Inlet d_e : Not read Outlet d_o = Not read

DISCHARGE : Observed weir head h = 0.99'
Add for approach velocity + .02
Corrected head 1.01 Q = 102 c.f.s.

$Q/D^{5/2}$ = 1.82 Critical depth d_c = 2.8'

PIEZOMETERS Nº	1	2	3	4	5	6	7	8
Reading	3.8	3.1	2.85	2.45	1.9	1.5	1.3	1.0
Calc'd depth	3.45	3.05	3.1	2.95	2.7	2.55	2.65	2.65

SPECIAL OBSERVATIONS : ? correctness of piezometer readings

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2.8	12.0	1.23	8.5	0.167	0.036

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
2.8	2.0	1.12	1.8

FLOW PROFILE :

Assumed uniform critical, d = 2.8'

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : G 1 LENGTH : 110' net INLET : Bevel flush
DIA. : 84" SLOPE : 1.4% $D^{5/2}$ = 130

OBSERVED DEPTHS : Headwater H : 2.5' H/D = 0.36
Tailwater T : Free overfall
Inlet d_e : Not read Outlet d_o = 2.1'

DISCHARGE : Observed weir head h = 0.67'
Add for approach velocity + .01
Corrected head 0.68 Q = 56 c.f.s.

$Q/D^{5/2}$ = 0.43 Critical depth d_c = 1.9'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading								
Calc'd depth								

SPECIAL OBSERVATIONS : Uniform depth through culvert

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
2.1'	9.7	1.13	5.78	0.119	0.035

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$

FLOW PROFILE :

Assumed uniform sub-critical, d = 2.1'

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : G2 LENGTH : 110' net INLET Bevel flush
DIA. : 84" SLOPE : 1.4% , $D^{5/2}$ = 130

OBSERVED DEPTHS : Headwater H : 4.1' H/D = 0.58
Tailwater T : Free overfall
Inlet d_e : not read Outlet d_o = 2.4'

DISCHARGE : Observed weir head h = 1.07
Add for approach velocity + .01
Corrected head 1.08 Q = 96 c.f.s.
Tailwater 0.5' above crest : correction factor 0.86
$Q/D^{5/2}$ = 0.74 Critical depth d_c = 2.5'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading								
Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
		Insufficient data			

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
	Insufficient data		

FLOW PROFILE :

Insufficient data.

CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : G3 LENGTH : 140' net INLET Bevel flush
DIA. : 84" SLOPE : 1.4% $D^{5/2}$ = 130

OBSERVED DEPTHS : Headwater H : 4.8' H/D = 0.69
Tailwater T : Free overfall
Inlet d_e : 3.8' Outlet d_o = 2.8'

DISCHARGE : Observed weir head h = 1.20
Add for approach velocity + .03
Corrected head 1.23 Q = 136 c.f.s.

$Q/D^{5/2}$ = 1.05 Critical depth d_c = 3.0'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading								
	Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$
$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3.0	15.9	1.36	1.13	4.13					
3.2	17.2	1.40	0.97	4.17	0.04	.0230	.0090	4'	4
3.4	18.8	1.44	.81	4.21	0.04	.0182	.0042	10'	14
3.5	19.4	1.46	.76	4.26	0.05	.0166	.0026	19'	33
3.6	20.1	1.48	.71	4.31	0.05	.0150	.0010	50	83
3.7	20.8	1.49	.66	4.36	0.05	.0138	0	NORMAL DEPTH	∞

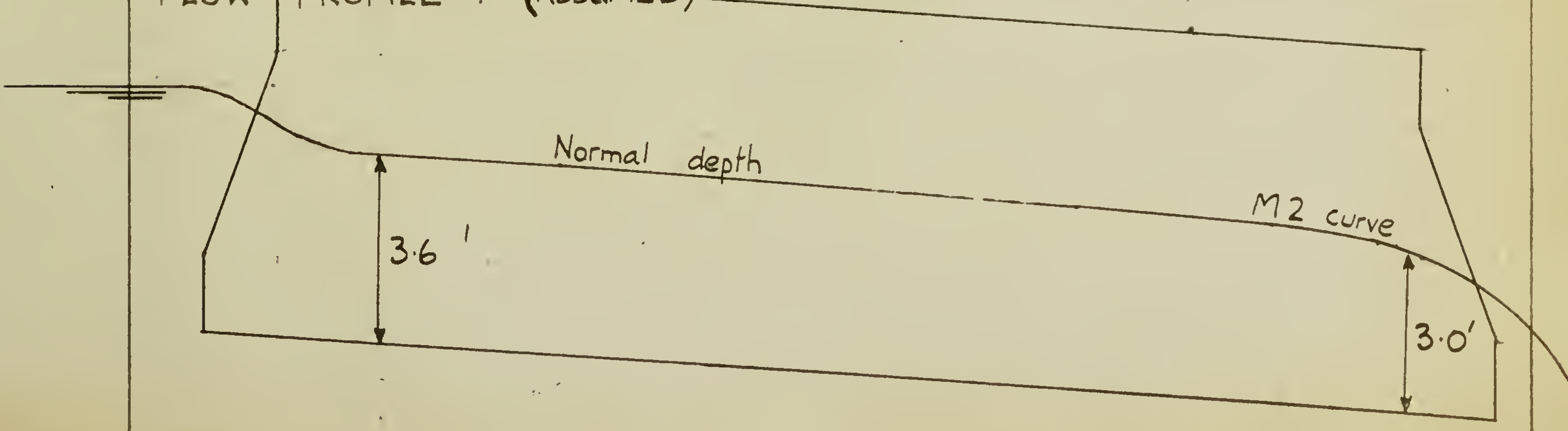
UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
3.6	20.1	1.48	6.77	0.118	0.038

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3.6	1.2	0.71	1.7

FLOW PROFILE : (ASSUMED)



CULVERT FIELD TESTS – DATA & ANALYSIS

TEST Nº : G4	LENGTH : 110' net	INLET : Bevel flush	
DIA. : 84"	SLOPE : 1.4%	$D^{5/2}$ = 130	

OBSERVED DEPTHS : Headwater H : 5.1' H/D = 0.73

Tailwater T : Free overfall

Inlet d_e : 3.5' Outlet d_o = 3.0'

DISCHARGE : Observed weir head h = 1.27'

Add for approach velocity + .03

Corrected head 1.30 Q = 148 c.f.s.

$Q/D^{5/2}$ = 1.14 Critical depth d_c = 3.1'

PIEZOMETERS

Nº	1	2	3	4	5	6	7	8
Reading								
Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$

$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L
3.2	17.2	1.40	1.15	4.35					
3.4	18.8	1.44	0.96	4.36	.01	.0165	.0025	4	4
3.5	19.4	1.46	.90	4.40	.04	.0151	.0011	36	40
3.6	20.1	1.48	.84	4.44	.04	.0137	0		∞

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
3.6	20.1	1.48	7.37	0.118	0.035

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
3.6	1.5	0.84	1.8

FLOW PROFILE :

Similar to G3

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : H1 LENGTH : 56' net INLET : Bevel flush

DIA. : 84" SLOPE : 0.6% $D^{5/2}$ = 130

OBSERVED DEPTHS : Headwater H : 2.5' H/D = 0.36

Tailwater T : Free overfall

Inlet d_e : Not read Outlet d_o = 2.1'

DISCHARGE : Observed weir head h = 0.67

Add for approach velocity + .01

Corrected head .68 Q = 56 c.f.s.

$Q/D^{5/2}$ = 0.43 Critical depth d_c = 1.9'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading								
	Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION

Assumed n = S_f = $/(AR^{2/3})^2$

$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
	Insufficient	data			

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$

FLOW PROFILE : Insufficient data.

CULVERT FIELD TESTS - DATA & ANALYSIS

TEST Nº : H2 LENGTH : 56' net INLET : Bevel flush

DIA. : 84" SLOPE : 0·6% $D^{5/2}$ = 130

OBSERVED DEPTHS : Headwater H : 3·9' H/D = 0·56

Tailwater T : Free overfall

Inlet d_e : Not read Outlet d_o = 2·7'

DISCHARGE : Observed weir head h = 1·07'

Add for approach velocity + ·01

Corrected head 1·08 Q = 96 c.f.s.

T.W. 0·5' above crest : correction factor 0·86

$Q/D^{5/2}$ = 0·74 Critical depth d_c = 2·5'

PIEZOMETERS	Nº	1	2	3	4	5	6	7	8
	Reading								
	Calc'd depth								

SPECIAL OBSERVATIONS :

BACKWATER CALCULATION : Assumed n = S_f = $/(AR^{2/3})^2$

$V^2/2g$ = $/A^2$

d	A	$R^{2/3}$	$V^2/2g$	E_f	ΔE_f	S_f	$S_o - S_f$	ΔL	L

UNIFORM FLOW CALCULATION :

d	A	$R^{2/3}$	V	$S^{1/2}$	$n = \frac{1.49 R^{2/3} S^{1/2}}{V}$
		Insufficient	data		

ENTRANCE DRAWDOWN CALCULATION :

d_e'	$h_e' = H - d_e'$	$V_e^2/2g$	$k = h_e' \div V_e^2/2g$
	Insufficient	data	

FLOW PROFILE :

Insufficient data